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W. Jarocki

Computation of Bridge Spans and Culverts

(Obliczanie otworów mostów i przepustów)

Published for the Department of the Interior pursuant to
an Agreement with the National Science Foundation, Washington, D.C. by
THE SCIENTIFIC PUBLICATIONS FOREIGN
COOPERATION CENTER OF
THE CENTRAL INSTITUTE FOR SCIENTIFIC,
TECHNICAL AND ECONOMIC INFORMATION

Warsaw, Poland
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AUTHOR'S FOREWORD

The number of postwar publications dealing with the computation of spans of bridges and culverts is limited. No officially valid regulations in this respect have been published thus far. Designers for the most part make use of prewar literature and continue to compute by obsolete methods.

Bridges and culverts should be computed for discharges with varying probability of appearance, since this is advantageous for the national economy. For this reason, the methods, described in prewar literature which do not take into account the frequency of hydrological phenomena, should not be used.

In order to compute the openings of structures for discharges with varying probability of appearance, some designers endeavor to use empirical formulas as given by Soviet literature. This involves difficulties, however, because the coefficients applied to these formulas are valid for the territory of the USSR and cannot always yield positive results for the rivers of this country.

The present book will enable designers to compute the openings of bridges and culverts for discharges with varying probability of appearance since I have taken into account Polish and Soviet formulas, the latter adapted to the conditions prevailing on Polish territories.

In order to adapt the most suitable coefficients to the formulas, designers should know the essence of the phenomena exerting their influence on the origin and development of water elevations. For this reason, I have presented the general characteristics of these phenomena prior to discussing the metods of determining the maximum discharges.

Furthermore I have elaborated directives with which the particular appendices to hydrotechnical calculation of bridge span building should comply, since there are yet no detailed regulations concerned with the preparation of these plans and the problem has hitherto been individually treated by every designer.

The methods of hydraulic computation of openings are presented separately for large bridges, small bridges and culverts. In discussing the culverts, considerable

space is deliberately devoted to the computation of the rectangular and round pressure openings, since the application of these types of openings can involve considerable economies, particularly when computations take into account the accumulation of water above the structure.

I have tried to present the methods of all the computations in a manner most convenient for practical use, and have eschewed intricate formulas, which can easily be found in appropriate textbooks. To facilitate the work of designers, tables and diagrams are added to the various formulas and examples of computations are elaborated.

I hope that this book will achieve its object, which is to make it easier for technical personnel to use the latest methods when computing the openings of bridges and culverts from field observations and laboratory investigations.

INTRODUCTION

From time immemorial, various types of structures have been built on rivers for the purpose of contacts and exchange of goods.

In the earliest times, such structures were built in a most simple manner, for example, piling up stones in the water, felling trees across rivers, etc. The building of ferries, footbridges and, subsequently, road bridges of primitive construction was started later.

The Egyptian tombs, Chinese miniatures and various ancient excavations bear witness to the fact that large bridges were built as long as 4,000 years ago, and small bridges even earlier. In the Roman era were built the famous Roman and Byzantine aqueducts to a height of up to 50 *m and a length of up to several thousand meters.*

At present, bridges and culverts of various kinds are built for transport purposes over waterways. The division of these structures into culverts and bridges does not represent any definitely established standard length or construction in the present suggestion of division. Thus, for instance, every small road structure serving for the passage of water is, regardless of its construction, called by some authors a culvert. There are, however, certain differences of opinion as to how long a structure should be in order to be reckoned among the small.

On the other hand, many authors maintain that all road structures used for allowing passage to water should be called culverts if they are bounded at top and sides by an embankment. Since the latter definition seems to be the most descriptive, we shall hereinafter distinguish bridges from culverts by the way they are located in road or railway embankment irrespective of span.

Formerly, no computations were made for the size of bridges and culverts. The height of the water level and the width of valley flats were simply taken into account. Therefore, even in ancient times, observations of the water level in streams were started, and they were not only helpful in determining the number of openings, but served also for other purposes.

It is well-known, for instance, that regular observations of the water stages on the Nile Rivers during freshets were conducted by Egyptians as much as 4,000 years ago. The water marks on a tide staff located in the Second Cataract of the Nile were primarily used for determining the limits of flooding the valley of this river so rich in fertile silts. The estimation of the yield of crops per unit of surface area of the entire country was also facilitated by these water marks.

The elaboration of various methods of computing maximum discharges was started along with the development of hydrology. Note that hydrological investigations have for long been conducted, because water has always been one of the most important factors in economic life. Hydrological problems even entered into certain works by Aristotle and many other scientists of antiquity.

The explanation of the correlation and cooperation of all phenomena dealt with by hydrology constitutes the main task of this science (as well as other sciences). It has been shown, however, by an analysis of various hydrological processes, that such do not appear regularly and never repeat accurately. To establish general rules governing hydrological phenomena is, therefore, difficult and requires prolonged investigations and observations. Hydrologists cannot therefore yet explain, foresee and relate one to another many hydrological phenomena occurring in nature.

Nevertheless, such elaborations are necessary to facilitate the establishment of the conditions of discharge within the limits of structures built on rivers, because expensive hydrotechnical equipment can be destroyed by flood if a faulty assumption had been made for the quantity of water discharge.

To meet these requirements, approximate formulas based on many empirical correlations are derived by hydrologists, since to date there has been no possibility of presenting the exact values of discharges lacking certain direct measurements and observations.

In view of the large number of bridges and culverts built, the entire work should be aimed at the most accurate computation of the size of openings, since that exerts a decisive influence on costs involved in such structures. Building too large an opening is uneconomical, while a structure with an insufficiently large opening can be destroyed by flood, involving losses caused not only by the necessity of reconstruction, but also by traffic hold-ups.

Particular attention should be given, therefore, to the elaboration of accurate methods of establishing maximum discharges and computing sizes of openings. Establishing maximum discharges should be primarily based on field observations and measurements. For the time being, however, we are sometimes compelled by the insufficient number of measurements to use empirical formulas. Nevertheless, efforts should be made to use formulas which are the most appropriate under given conditions.

The size of structure openings depends to a considerable extent on the shape of their inlets and outlets, distribution of the regulating structures, direction of the currents of a highwater, etc. Investigations and observations connected with the

work of structures should therefore be conducted under various conditions, the theoretical bases being derived for individual cases.

This will facilitate the determination of the mutual reaction of stream, channel and structure as also of a quantitative and qualitative definition of phenomena — factors which will considerably improve the value of computations.

More accurate results can be obtained by means of laboratory investigations, which are a valuable supplement to theoretical work and field observations. Laboratory investigations conducted on models of bridges and culverts facilitate economic design of such structures having the most advantageous shapes from a hydraulic point of view.

Sometimes, theoretical considerations fail when action is simultaneously exerted on a structure by various factors; the correct method of solving difficult problems in such cases can be ascertained only by laboratory investigations conducted on models.

CHAPTER I

GENERAL INFORMATION

1. Fundamental Definitions

The development of hydrology has necessitated introducing many new concepts and definitions to facilitate the discussion of phenomena under investigation.

The following names and definitions used in hydrological computations are the most important.

Catchment basin (or catchment area or river basin) is an area drained of water by a river; catchment basins of small streams are sometimes called drainage basins (or areas).

The discharge Q is a quantity of water flowing through the cross section at a given moment per unit of time; it is commonly expressed in cu m/sec;

$$Q = Fv$$

where:

F — surface of a cross section in sq m

v — mean velocity of flowing water in m/sec

Unit runoff q is the discharge per unit of surface of a catchment basin. The runoff is usually expressed in liters per second per square kilometer:

$$q = \frac{Q}{A}$$

where:

Q — volume of discharge in l/sec

A — catchment basin surface in sq km

The annual volume discharge W or the total volume of water flowing through a river cross section per annum is customarily expressed in cubic meters or cubic kilometers.

The mean annual discharge Q_0 in cu m per second is a ratio of the annual volume discharge W to the number of seconds in a year T:

$$Q_0 = \frac{W}{T}$$

The year usually consists of 365 days and therefore, $T = 31{,}536{,}000$ seconds.

The modulus of runoff M or the mean annual discharge per unit of the catchment area surface is generally expressed in liters per second per square kilometer:

$$M = \frac{1{,}000\, Q_0}{A}$$

The standard of runoff or the mean modulus of runoff M_0 in liters per second per sq km is a sum of the moduli of runoff during n years divided by the number of these years:

$$M_0 = \frac{M}{n} \text{ l/sq km}$$

The height of the annual layer of runoff t or the height in mm of the uniformly distributed annual volume discharge W on the catchment basin surface A, is computed by the following formula:

$$t = \frac{W}{1{,}000\, A}$$

where A is expressed in square kilometers and W — in cubic meters.

The coefficient of volume discharge α is either the ratio of the volume runoff during a certain period to the volume of rainfall occurring during the same period in a given area, or the ratio of the layer height (in mm) of flow to the height h (in mm), of the precipitation layer during an identical period

$$\alpha = \frac{t}{h}$$

The coefficient of the runoff modulus k is the ratio of volume discharge W during a certain period (e.g. a year) to the magnitude of the mean volume discharge W_0 for the same period computed over a number of years:

$$k = \frac{W}{W_0}$$

The coefficient of the runoff modulus K can also be determined by other values, e.g.:

$$k = \frac{M}{M_0} = \frac{Q}{Q_0} = \frac{t}{t_0}$$

The discharge hydrograph is a diagram of discharge fluctuation in correlation with time. In preparing this graph, the discharge volumes are plotted on the vertical and the time on the horizontal axis. The surface of the hydrograph drawn for the annual period, multiplied by the number of seconds in a 24-hour period, yields the volume of the annual flow.

A period, in which changes are caused in the natural regimen of rivers by

changes in meteorological conditions is called a hydrological or water year. The beginning and end of a water year, which lasts twelve months, do not usually coincide with the beginning and end of a calendar year.

The moment in which a turn in the natural feeding of a river takes place can be regarded as the beginning of a water year. Such a moment appears when rainfalls cause the water stages of a river to rise. Up to this moment, a river is fed almost exclusively by the flow of ground water and, therefore, it is marked by low water stages.

The turn in the natural feeding of rivers does not occur everywhere at the same time. It depends on local climatic conditions. This explains why the water year can begin at various periods not only in various countries, but also in particular areas of the same country. For instance, various dates mark the beginning of the water year in individual areas of the vast territory of the Soviet Union.

In Poland, November 1 is adopted as the beginning of the water year throughout the country. The water year is divided into the two 6-month periods: winter months — from November 1 to the following April 30, and summer months — from May 1 to October 31.

The turn in the natural feeding even of the same river can in individual years, take place at different periods. Nevertheless the same conventional date for all years is always adopted as the beginning of a water year.

It should be borne in mind, however, that meteorological phenomena appearing during one of the water years may exert an influence extending far into the subsequent years. For instance, surface water originating from the rains fallen at the end of a water year can rapidly run off to rivers, while the subsoil water formed by the same precipitation can feed rivers for a long time. For this reason, hydrological periods extending more than one year should be taken into account to obtain a more accurate correlation betwen precipitation and flow.

2. General Principles for Computing Spans of Bridges

The computation of spans of bridges and openings of culverts primarily consists in determining the maximum volume of a discharge and the size of opening required to pass it through. Determining the former magnitude is a hydrological — and the latter a hydraulic problem.

Thus far no guiding rules exist to explain beyond any doubt which of the discharges should be called the highest and which maximum discharge, peak discharge or flood wave. In practice, these names have been used to denote the discharge occurring at the highest water stage observed. If there were no observation stations on some streams, the highest water stage was determined from evidence given by local residents.

The discharge volume is a product of a river cross section and water velocity. As it is difficult for many reasons to take a direct measurement of velocity

at the highest water stage the velocity is computed by empirical formulas, mostly the Chezy formula:

$$v = C\sqrt{Ri}$$

This formula facilitates the computation of the mean velocity by means of the measured slope of the water surface i and the hydraulic radius R. The coefficient C appearing in this formula is computed by various equations the number of which now reaches a hundred.

As already indicated, besides the velocity, the stream cross section should also be known for the determination of a discharge. However, the cross section of some streams, especially the small ones is not measured. In such cases, empirical formulas are used, which facilitate the computation of maximum discharges without using velocity and cross section; other elements and properties of a stream are then taken into account, primarily the size of a catchment area and magnitude of precipitation.

The discharge established by the above mentioned methods and called maximum discharge, peak discharge or a flood wave, is actually the highest during the period of observation or during the period from which materials were taken for deriving empirical formulas. However, the mumber of the observation years, magnitude of precipitation, etc. usually vary not only oni ndividual streams, but also over different sectors of the same stream.

The highest water stage is thus established at random and the probability of its occurrence is variable.

After the heavy losses caused by the collapse of dams in the USA and Italy in the early 20th century, hydrologists came to the conclusion that hydrotechnical structures cannot be computed for the water stage established at random but water levels and discharges which only appear very seldom should be taken into account.

For the same reasons, computing spans of bridges and openings of culverts for the highest discharges established incidently is incorrect, but discharges with a defined probability of appearance should be taken into account for the determination of which a method of mathematical statistics is used.

After establishing a reliable discharge with an adopted probability of appearance, hydrologists begin to compute the size of an opening. This computation consists in choosing the cross section of an opening sufficient for a safe passage of a maximum discharge with the adopted probability of appearance.

A bridge site should be selected so as to be most suitable from the geological, hydrological and economic standpoints as well as the best as regards location. Then a reliable mean water velocity under the bridge is adopted equal to the mean natural velocity in the main channel having no structures.

Since the length of a bridge decreases with the increase in velocity, the velocity under the bridge can sometimes be adopted somewhat lower than the

natural velocity in the main channel. In such a case river bed scour to a certain depth under the bridge is admitted or artificial deepening is applied.

The discharge of water through the openings of small bridges and culverts can take place when the (Fig. 1) inlet of a structure is flooded or free. A free opening does not operate with its full cross section, and it passes a smaller quantity of water than does a flooded opening. The rise in the velocity of water flowing through the pressure opening is caused by the elevation of the water above the structure with the flooded inlet.

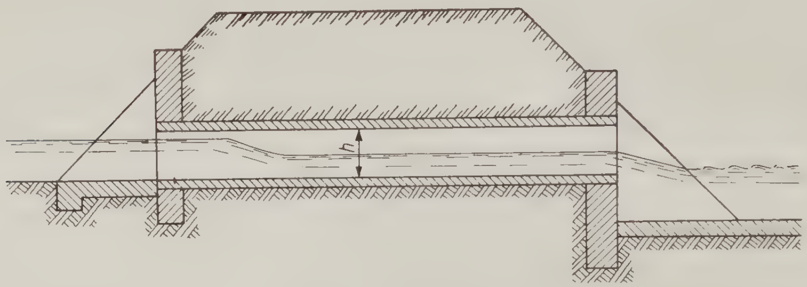

Fig. 1. Culvert with a free intake

It has been assumed thus far, that, when the inlet is entirely submerged the opening always works with its full cross section. Recent investigations have shown, however, that, if the inlet edges are not streamlined the pressure opening can also operate when its cross section is not entirely filled. For this reason, the method of computing small openings depends not only on the inlet being submerged or free, but also on the shape of the inlet (which can be hydraulically favorable or unfavorable).

Also to be taken into consideration is the fact that openings computed as free can in fact be flooded and operate under pressure. In this connection such a size of opening should be adopted as will not allow of any damage to the structure as a result of additional pressure resulting from a possible flooding.

3. Hydrological Publications

More accurate results can be achieved in computing bridge spans by using existing observation data and available materials.

The oldest observation of water stages in Poland were conducted by Magier, a professor of the Warsaw University, who recorded from 1799 the daily water stages on the Vistula River near Warsaw.

It was in the same year that the water gaging station was set in operation on the Vistula at the Montawski Promontory. The Cracow water gaging station

was founded in 1813. In 1818, observations of water stages were started in Poznań on the Warta River and in 1823 — on the Noteć River.

Information concerning the old observations on the territory of the former Congress Kingdom is presented in „Wisła, jej bieg, własności i spławność" (The Vistula: Its Route, Characteristics and Navigability) by Kolberg, Part II, Warsaw 1861, and in „Pamiętnik Fizjograficzny" (Physiographical Diary), Vol. I. published in 1881.

According to Kolberg, in 1838 a water gaging station was opened in Zawichost, and in 1847 nine more water gaging stations were put into operation on the boundary sector of the Vistula. The investments were made from funds allocated for building highwater dikes. These stations — except for Zawichost and Korczyn — were closed down in 1861.

The height of the zero points on staff gages are given in Kolberg's work, together with water stage observations of the Zawichost gage between 1841 and 1860 and many other data.

In 1877, regular observations were already being conducted at ten water gaging stations located within the boundaries of the former Congress Kingdom. The tables containing data of daily observations, together with the characteristic water stages and diagrams of water level fluctuations on this territory between 1876 and 1910 were published by the former Ministry of Communication of the Russian Empire in Petrograd.

The results of observations of water stages on the staff gages from Niepołomice to Zawichost up to the middle of 1914 were sent to the Hydrographical Bureau in Lwów and published in the hydrographical yearbook issued by that institution.

Measurements of the volume of discharge on the rivers of the former Galicia province were started in 1897. The results were published in the Hydrographical Yearbook of the Central Hydrographical Bureau in Vienna.

Measurements of discharge were later started on the territory of the former Kingdom of Poland. The results of only a few measurements of discharge appropriate to the period between 1904 and 1908 are known.

As regards disastrous floods on the territory of Poland, the oldest information in this respect dates back to August, 1813. A violent flood then occurred embracing, besides Poland, the territories of Hungary, Czechoslovakia and a part of Germany.

A flood causing the absolute maximum water stage on the Vistula in Warsaw (655 cm) and on some mountain rivers appeared in July 1844.

Unusually high water stages were caused on the San by exceptionally violent rains in the central sector of the Carpathian Mountains in July, 1867. The rise on the San, together with the flood wave of the River Dunajec had a particularly strong influence on the water level of the central part of the River Vistula. The highest stages ever recorded on the River Bug and in part of the River Narew basin were caused by the 1888 flood.

Very high water stages in Cracow and in other cross sections of the Upper Vistula up to the mouth of the River San as well as on the Soła, Skawa, Raba and Dunajec occurred in July, 1903.

The June 1925 flood was particularly violent and caused especially heavy losses in parts of the Silesia, Cracow and Lwów regions.

Substantial losses were also involved in the flood that took place in July, 1934. Intensive rainfalls (e.g. at Witów — 285 mm in a day) caused a rise in water stages by several meters in a few hours on the Carpathian tributaries of the Vistula. Particularly formidable was the course of this flood in the river basin of the Upper Vistula and on the Dunajec, Raba, Skawa and Wisłoka rivers where a considerable excess was recorded above the highest stages formerly observed. It should be mentioned, however, that the crest of the 1934 flood was lower than the crests of 1813, 1839, 1844, 1845 and 1867.

Publication of the Results of Observation at the Water Gaging Stations

Parts of the catchment areas of the Vistula, Oder, Niemen and Danube rivers, and river basins along the sea coast situated on Polish territories are covered by the present observation network.

The water gaging stations, whose observations are published in the Hydrographical Yearbooks issued separately for each year and each river, are located in the main cross sections of streams belonging to these catchment areas.

The hydrographical Yearbooks usually contain the following material:

(a) daily observations of water stages in the main water gaging cross sections, together with the extreme and average stages and the location of the water gage station, river basin surface and elevation of the gage zero point,
(b) list of water temperatures;
(c) list of characteristic stages of the ground waters;
(d) collection of measurement results taken of the discharge in a given year;
(e) collection of data on precipitation for the most important stations;
(f) general survey of hydrological conditions;
(g) diagrams of the water level fluctuations,
(h) ice phenomena,
(i) diagrams of the course of changes in the ground water levels;
(j) map of the catchment area with the location of the station marked on it.

Observations on the water levels of the Vistula river basin situated within the boundaries of the former Austrian occupation are presented in the 1913 to 1918 hydrographical yearbooks published by the Polish Hydrographical Service.

The yearbooks for the entire river Vistula basin were published for 1919—1937, and for the river Oder basin within the prewar boundaries for 1919—1932. The publication of the yearbooks for the Vistula and Oder river basin within the present boundaries of Poland was started gradually after the last war.

Furthermore, observations on water levels in Polish territories for the years between 1900 and 1940 can be found in the German yearbooks entitled „Jahrbuch für die Gewässerkunde Norddeutschlands".

These yearbooks are divided into a general part, in which are given an alphabetical index of the water gaging stations, a review of fluctuations in water stages and of ice phenomena and into six sections, containing daily observations of water stages in the individual cross sections together with the results of measurements of discharge and characteristic stages.

Polish territories are included in Sections I, II and VI. Section I includes the Niemen, Pregoła and Vistula river basins, section II deals with the River Oder and section VI — with the rivers of the coastal region.

Between 1900 and 1914, the daily observation data appropriate to the water gaging stations located on the territories then occupied by Prussia, the main water gaging stations located in the regions occupied by Austria (e.g. Cracow and Chwałowice on the River Vistula, some cross sections on the Dunajec, San and other rivers) and by Russia (Warsaw, Zakroczym, etc.) were recorded in the part of Section I of these yearbooks dealing with the River Vistula basin.

From 1914, the number of stations in the River Vistula basin for which observation data were recorded, was gradually diminished; as late as 1928, however, the observations of certain Polish stations — e. g. in Toruń — were published in these yearbooks.

From 1929 to 1940 the observations concerned only the water gaging stations situated within the boundaries of Prussia and Gdańsk.

The results of observations and measurements published up to 1917 in Section II concerned the River Oder basin within the boundaries of the region occupied by Prussia, while those published from 1918 onwards applied to the area within the German frontiers after World War I.

Observation data from the meteorological stations were published before the last war in the yearbooks of the Polish Meteorological Institute — divided into several parts. The height of precipitation for the stations located in the catchment areas of all the rivers situated on the territory of Poland was published in Part III of these yearbooks.

Before the last war, the yearbooks of the Polish Meteorological Institute were published for the years 1919—1920 and 1925—1933. For the periods 1921—1924 and 1933—1937, which were not covered by the meteorological yearbooks, the data on observations and precipitation can for the most important stations be obtained from the hydrographical yearbooks.

The German publication "Ergebnisse der Niederschlagsbeobachtungen" should be used for computing precipitation in the years before the war for the present Western Territories of Poland.

Precipitation data have been published since the war in the Precipitation Yearbooks of the Polish Hydro-Meteorological Institute.

Publications Containing Lists of Characteristic Stages and Discharges

The lists of stages and discharges characteristic for individual rivers are published in the "Proceedings of the Polish Hydro-Meteorological Institute".

Each issue of this series of publications contains general information on a stream — together with the map of its catchment area and the following data for each important cross section:

(a) description of a cross section with a water gage and changes of the elevation of its zero point on the gage;
(b) list of water stages, including extreme and average stages;
(c) list of ice phenomena;
(d) enumeration of the measurements made of water discharge;
(e) discharge curves;
(f) list of the characteristic stages and discharges;
(g) table of uniform discharges together with diagrams.

Publication of the Results of the Measurements of Discharge

The results of the measurements of discharge in the catchment area of the Dunajec and San, as well as rivers covered with ice, were published before the war in this series of works.

During the war, two works of this series were published: "The Results of the Measurements of Discharge Conducted in the Oder River Basin Between 1888 and 1937", and "Die Ergebnisse der Abflussmengenmessungen der Weichsel 1836—1939".

The first mentioned work dealing with the Oder river basin is divided into two parts. The first of these covers the period between 1919 and 1937, the second, between 1888 and 1915.

Each part includes:

(a) results of discharge measurements on the Warta river;
(b) results of discharge measurements in the right-bank section of the Warta river basin;
(c) results of the discharge measurements in the left-bank section of the Warta river basin;
(d) results of the measurements in the Upper Oder river basin.

The results of discharge measurements in the cross sections of the River Vistula for the periods between 1919 and 1939 and 1836—1918 are presented in the work discussing this river.

The following works dealing with measurements of the discharge volume were published since the war: "The Results of the Discharge Measurements in the Left-bank Section of the Vistula River Basin Between 1895 and 1939" and "The Results of Discharge Measurements Taken in the Mountain Tributaries of the Vistula Between 1899 and 1939".

Publications Containing Computations of the Surface of the Catchment Basin

This series of publications usually contains the following material:
(a) general part with a description of the methods worked out;
(b) enumeration of the computations of the surface of the catchment basin;
(c) list of maps;
(d) hydrographical map of a catchment basin.

The detailed division of the catchment basins of the San, Pilica, Gostynka, Przemsza, Chechła, Niemen and Dvina rivers were published before the war.

The following works dealing with determination of the surface of the catchment basin have been published since the war: "The Detailed Division of the Oder River and Coastal River Basins", "Materials for the Hydrography of the Upper Orawa River Basin" and "The Detailed Division of the Vistula River Basin".

Publications on Water Balance

Materials concerning precipitation and discharges in particular river cross sections are published successively in the Proceedings of the Polish Hydro-Meteorological Institute called "Materials for the Water Balance in Poland".

Other Sources and Publications

The results of observations and measurements conducted in certain river cross sections up to 1910 can be found in the 1921 publication entitled "Materials on the Hydrography of the Former Congress Kingdom".

This work appeared in two separate numbers. Number I contains data concerning observations of water stages and elevations on the staff gages on the left bank of the Vistula between Niepołomice and Zawichost, together with a description of the measurements of discharge taken on the River Vistula near Sandomierz. Number II contains data on the highest and lowest water stages and on thawing and freezing of certain rivers during the period between 1881 and 1910, according to the observations of the water gaging stations.

The characteristic water stages and ice phenonema from 1901 to 1910 and the results of the measurements of discharge taken on the Vistula near Sandomierz in 1908, together with diagrams of the hydrometric cross sections for the main water gages on the Vistula, are presented in Number I.

The highest and lowest water stages and dates of the ice phenomena from 1881 to 1910 for 15 water gaging stations on the Vistula, 2 stations of the Nida, 1 station on the Tanew, 8 stations on the Bug, 1 station on the Biebrza, 5 stations on the Narew and some dozen stations on the Niemen, Szczara and Wilia rivers are given in Section II.

Auxiliary materials which may be helpful for studies for and computing bridge spans can further be found in the "Hydrological Service News", "Hydro-

-Meteorological Service News", "Proceedings of the Polish Hydro-Meteorological Institute" and "Hydro-Meteorological Review".

Particularly noteworthy among several valuable articles published in these periodicals is the work: "Characteristic Water Stages and Discharges in the Water Gaging Stations in Cross Sections of the Vistula River" by S. Siebauer, "Hydro-Meteorological Service News", vol. I, section I, 1947.

The catchment basin surfaces, heights of the zeros on water stage gages up to 1941 and after 1941, and also discharges and falls of water from the highest to the lowest water stages, are shown in this work for particular cross sections on the River Vistula.

CHAPTER II

FACTORS INFLUENCING THE DISCHARGE

The surface of the geoid-shaped earth amounts to about 510,000,000 sq km of which oceans and seas account for about 70.8 percent i. e. 361,000,000 sq km.

The water resources of the earth are expressed by the figure of 1,304,068,550 cu km which does not include water contained in chemical compounds.

Under the influence of the action of sunrays, water in the form of vapor

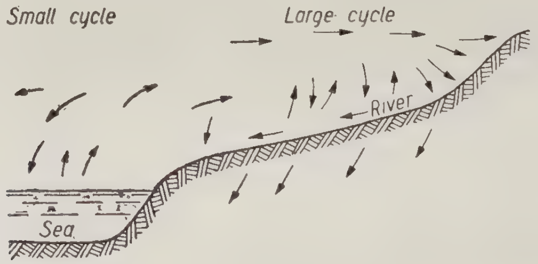

Fig. 2. Hydrologic cycle

soars over the earth's surface and is sometimes transported for considerable distances by air currents. Under favorable conditions, vapor concentrates and falls to the earth in the form of rain, snow or hail. This phenomenon is continually being received on the surface of the earth.

The vapor formed over oceans and seas is carried by humid winds to the land, while a considerable part of it, after being condensed, falls in the vicinity of seas and oceans mostly in the form of rain; the rest of it being transported further inland (Fig. 2).

Of the water falling on the surface of the land, part evaporates, part runs off to the streams and part sinks into the soil, seeping through it to the rivers or once more emerging on the surface of the earth in other places in the form of springs.

Two types of water circulation — great and small — are distinguished in nature. Small circulation arises when water evaporating from the ocean surface once more reaches an ocean falling there in the form of precipitation. Great circulation, on the other hand, consists in the fact that some part of the vapor is carried further on by humid winds and falls on the surface of land. A part of this precipitation does not evaporate, but returns by way of rivers to seas and oceans, thus performing what is called the great circulation.

It is assumed that:

(a) the annual amount of water evaporating from seas and oceans is equal to the sum of precipitation falling onto their surface and the quantity of water fed to them by rivers;

(b) the annual amount of water evaporating from the earth's surface is equal to the difference between the quantity of water falling on to the earth and the quantity of water running off to the oceans.

The magnitude of precipitation and evaporation on the globe during one year as determined on the basis of investigations, is shown in Table 1.

Table 1

Annual Quantity of Precipitation and Evaporation

Name	Quantity of Precipitation and Evaporation in cu km		
	oceans	lands	total
Precipitation	412000	99000	511000
Evaporation	449000	62000	511000
Difference	−37000	+37000	—

Since the annual quantity of water falling onto the surface of the earth is equal to 511,000 cu km and the quantity of water contained in the atmosphere is equal to 12,300 cu km, therefore the water circulation must be repeated 41 times a year (511,000 : 12,300). As a result, the vapor contained in the atmosphere should traverse the entire route of circulation during 9 day cycles (365 : 41).

If the entire annual quantity of precipitation is expressed by the figure 511,000 cu km only a very insignificant percentage of the entire water resources take part in the general water circulation in nature — namely:

$$\frac{511{,}000}{1{,}304{,}068{,}550} \times 100 = 0.04 \text{ percent.}$$

Hydrological phenomena primarily depend on the physical state of water and conditions of its origin, together with accumulation and consumption both on the land surface and in the near distance above and below such surface.

When investigating hydrological phenomena an accurate examination should be made of the local conditions under which these phenomena appear, and the intercorrelation and various physiogeographical factors should be established.

There exist numerous and various factors exerting an influence on the discharge, whereas some of them lower the quantity of discharge, some others cause it to rise.

Of all the climatic elements, precipitation and evaporation exert the greatest influence on the volume of discharge.

The general regimen of a stream, formed under the influence of an existing climate, can be changed under the influence of the following additional factors:

(a) size and shape of a catchment basin;
(b) density of a river network;
(c) direction of the water movement in a catchment basin;
(d) type of soil;
(e) geological structure of a catchment basin terrain;
(f) appearance of lakes and marshes in a catchment basin;
(g) catchment basin vegetation cover;
(h) overgrowing of rivers;
(i) appearance of eternal snows, etc. in a catchment basin.

Furthermore, the shaping of the regimen of a stream is also affected by human activity which consists in building water elevating structures in a catchment basin, reclamation works, soil tillage, etc.

1. Climate

A collection of the meteorological elements characterizing the physical state of atmosphere over a short period of time is called the weather, while a certain continuity of meteorological phenomena caused by physical-geographical conditions and appearing over many years in a weather regime is determined by the climate of a given area.

Hydrological conditions are affected to a large extent by the climate, because the entire magnitude of discharge as well as the discharge during particular periods of a year depends on the quantity and type of precipitation, its distribution in time and space, evaporation, air humidity, wind velocity and other meteorological factors.

For quite a long time, some investigators sought to prove, on the basis of various premises and considerations, a hypothesis concerning the periodical or durable changes of climate appearing on the earth. This problem should be examined in detail and definite conclusions drawn, because it greatly influences

the hydrological computations. Bridges and culverts may endure for tens and even hundreds of years and, therefore, in the event of a possible change in the climate they should be adapted to the new conditions.

The adoption of a hypothesis concerning periodical changes in climate resulted primarily from the work published in 1890 by Professor Brueckner, of the University of Vienna.

The full duration of climatic changes was assumed by Brueckner, within limits of 30 to 40 years (average 35 years), whereas the period of 15-20 years includes the dry and warm phase which gives way to the cool and humid phase covering the subsequent 15-20 years.

At first, Brueckner's climatic change periods were adopted without reservations; it has been proved, however, by subsequent accurate research, that many inaccuracies and simplifications were introduced by Brueckner during his studies. Scientists dealing with the explanation of this problem arrived at no proof of the correctness of Brueckner's hypothesis. Polish meteorologists — Arctowski, Gorczyński and Gumiński — who also examined the question did not confirm the theory of cyclic changes in climate. Finally, it has been decided, that Brueckner's thesis of the existence of the 35-year period of climatic fluctuations is incorrect, or at least cannot be proved.

Some scientists tried to establish a correlation between climatic fluctuations and rhythmic changes in the number of spots on the surface of the sun. In this case, the existence of 11- to 12-year periods was mostly accepted, although there were also other opinions. However, in view of considerable discrepancies in results, inaccuracy of assumptions and inability to explain many phenomena, the hypothesis of this group of scientists, who suspected the correlation of climatic changes with the appearance or disappearance of sunspots, has not been confirmed either.

Other conclusions have been drawn on the basis of subsequent investigation of climatic changes. Thus, for instance, Bogolepov seeks to prove that there is no permanent change in the climate, but that there are fluctuations occurring every 11 and 33 years, and very stable ones every 100 years. The periods of these fluctuations can overlap each other and then particularly violent periodical climate fluctuations appear.

Since the latest investigations have not been able to prove periodical or stable changes of climate the following opinion now prevails among hydrologists and meteorologists:

(a) there were no periodical changes in climate during the latest few dozen decades,

(b) there have been no changes in climate since the last glacial epoch, which appeared 20,000 years ago.

The above principles have also been accepted by Soviet hydrologists, who have adopted them as a basis for all their computations.

It is to be assumed that the periodical changes in climate and humidity are possible in the event of a steady appearance of factors influencing the climate. These phenomena, however, have no cyclical features, are incidental and, with a different distribution of operative factors, their appearance may take an entirely opposite course.

Thus, for instance, the steady increase in the quantity of annual precipitation indicated by meteorological stations, has been ascribed to climatic changes in the adjoining areas. It has been shown, however, that this increase was caused by a change in existing local conditions, and particularly by the development of industry in the centers where observation stations were located. It has been stated, for instance, that on account of smoke, the quantity of precipitation is 10-30 percent higher in towns and industrial districts than in neighboring vicinities.

A fall in water level caused, for instance, by river alignment, building of water elevating structures, etc. does actually appear on some rivers. After long periods of time, when stabilization takes place, the original regimen is restored.

Grishanin[1]) in his work published in 1952 wrote: "The opinion concerning a supposed steady decrease in the depth of rivers, generally accepted by the scientific circles of the late 19th century, is incorrect so far as the magnitude of annual discharge volume is concerned. During the last 1,000 years annual flow was subject only to incidental fluctuations in one or other direction from its permanent mean value".

It is to be supposed, therefore, that the steppe-like character said to be assumed by some territories of Poland, which was believed to result from a higher temperature and a decrease in the quantity of precipitation, is actually apparent, temporary and caused by the influence of various additional factors not so far determined. Thus, for instance, 1952 was not similar to the previous dry years. In 1952, precipitation appeared almost every day, so that the ground waters which recently showed a very low level once more reached stages known a dozen or so years back. Violent floods to an extent unobserved for several centuries were produced by the spring of 1953.

If we assume that the steppe-like character does really appear, then we must, to be consequent, assume that there is now less probability of the highest discharges once observed which would necessitate decreasing the size of openings and spans computed. This cannot be resolved in the absence of sufficient data, and resolved only if the hypothesis concerning the supposed change of climate has a genuine basis, and if irrefutable proof in support are forthcoming in the future.

At present, however, the contention supported by the majority of scientists

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[1]) Grishanin: Rechnoi Potok (River Flow). Moskva 1952.

the world over, to the effect that there are no cyclic and permanent changes in climate, should be accepted as valid and reliable, and all hydrological computations should be based on it.

2. Precipitation

Vapor, together with air, is borne horizontally and vertically by air currents. Vapor does not change its form in air until in quantity it reaches the limits of saturation. Then, the condensation of vapor takes place and precipitation arises in the liquid (rain) or solid (snow and hail) form.

Vapor is condensed more rapidly in lower temperatures. Thus, for instance, at a temperature of 30°C, the quantity of uncondensed vapor in 1 cu m of air amounts to 0.5 g at 0°C, it increases to 5 g and at +30°C reaches 31 g.

In summer, clouds are formed by the warm masses of air being lifted to the upper cool layers of the atmosphere, where vapor is cooled and, after condensation, falls to the earth in the form of rain or hail according to conditions. The drops of rain are formed by the tiny droplets joining each other within the cloud.

When they reach such a size that they can no longer be held by the ascending air currents, they fall in the form of rain.

The junction of droplets takes place when a cloud cools as it is lifted upwards. If the temperature at the height to which the cloud is lifted is lower than 0°C, then the droplets turn into ice crystals and snow is formed.

If, at the same time, ice crystals and cooled droplets of water are contained in a cloud, then hail is formed.

Factors Influencing the Quantity of Precipitation

Generally used in hydrological computations is a layer of precipitation in millimeters fallen during the period of a year and is fixed as a mean for a series of years. It is called normal annual precipitation or standard precipitation. The lowest normal annual precipitation amounting to 3 mm appears in Chile, while the highest — 11,420 mm — is recorded in Bengal (India) and on the Hawaiian Islands.

The quantity of precipitation changes in relation with the elevation of the terrain type and direction of winds, and geographical situation of the area.

Usually, the quantity of precipitation rises together with the rise in the elevation of the terrain up to 3,000—4,000 m above sea level and then begins to fall (Fig. 3).

As an average, it may be assumed that the quantity of precipitation rises 20 percent per each 100 m of elevation of terrain. It should be remembered, however, that considerable deviations forming this principle may arise in some areas.

The quantity of precipitation is also affected by the direction of the movement of air masses. If air masses en route meet a mountain chain, precipitation is then formed, the quantity of which is higher on that side of the mountain from which the air is coming. Thus, for instance, Atlantic air masses cause precipitation of 2,300 mm a year on the Norwegian coasts, while the total annual precipitation in other parts of the Scandinavian peninsula behind the mountains, amounts to about 600 mm.

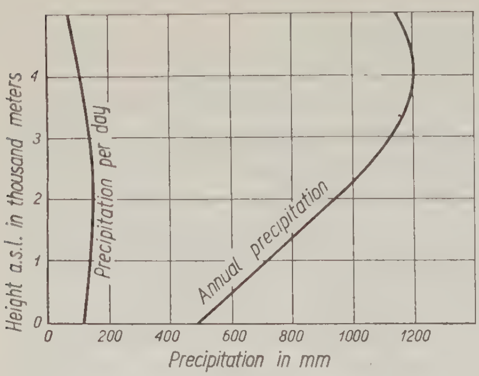

Fig. 3. Magnitude of precipitation, depending on elevation above sea level

The quantity of precipitation usually diminishes as the distance from the sea coast increases, although this is not an absolute rule.

Table 2 presents the precipitation standards for the territory of Poland, averaged on the basis of observations between 1891 and 1930.

Attention is drawn to the fact that the magnitude of the runoff depends not only on the quantity of precipitation, but also on its stages, appearance in time, air temperature, etc.

Snowfalls yield a higher coefficient of runoff than the summer precipitation because in summer water losses involved in evaporation are very high. If there are low quantities of rains, then water formed by them reaches the rivers at a slower pace and the runoff decreases, while during downpours the magnitude of runoff is greater.

The increase in runoff may be caused by precipitation not only in the near future, but also during several subsequent years. Catchment basins have the properties of a reservoir, in which precipitation water seeping through the soil is accumulated. This water runs off to a river in subsequent years.

Air temperature is also a very important factor. At low winter temperatures, the quantity of snow falling in the lowland catchment basins can be considerable and, in the spring, when temperature rises, it may cause an intensive runoff. On the other hand, in summer the runoff of the lowland rivers is usually insignificant, because water formed by considerable rains rapidly evaporates on account of high temperatures and only a small quantity reaches the rivers.

Intensity of Rainfalls

To compute openings for the rises caused by summer rainfalls, very often the quantity of precipitation and its duration must be established. On the other

hand, the intensity of thawing snow, and quantity of water contained in it before the beginning of thaw, must be determined in considering the spring rises.

The quantity of water in cubic meters flowing through the opening per one second is valid for the computation of a bridge span. Therefore that part of the precipitation which will reach a structure in a comparatively short time — and not the total quantity — must be determined.

In this connection, a notion of a mean intensity of precipitation (or mean strength) should be introduced, which is equal to the ratio of the precipitation height h in millimeters, to the time of its duration t in munutes:

$$i = \frac{h}{t} \text{ mm/min}$$

Many scientists have concerned themselves with establishing a correlation between the rain intensity and the time of its duration and the surface on which it falls. Thus, for instance, during the period between 1907 and 1914, this problem was examined on the territory of the Southern Ukraine by Dolgov who came to definite conclusions proved by subsequent investigations.

Important work to this effect has been performed by Häuser in Bavaria, where 800 precipitation observation stations are located, 150 of them equipped with pluviographs. Observation materials from these stations were supplemented by Häuser with 250,000 questionnaires which he collected over 10 years containing the beginning and end of the appearance of rain and the precipitation total.

This problem was recently examined by Bogomazova and Petrova who used material concerning 17,200 rains fallen on the territory of the USSR.

The following two principal properties of rain were established from their findings:

(a) intensity of rain increases together with the decrease in the time of its duration;
(b) rains with higher intensity cover a smaller surface of terrain.

Rains and Downpours

Rainfalls are usually divided into rains and downpours (heavy rains, torrential rains, rainstorms), rains with high intensity being classified as downpours. According to some opinions a downpour is a rain which yields 5 mm precipitation in 1 hour; according to others — 10 mm, 20 mm and 30 mm respectively. Rain yielding 12 mm precipitation in 1 hour is commonly adopted by authors of Polish publications as a downpour. The inadequacy of these definitions — over and above their considerable discrepancies — consists in the fact that precipitation of shorter or longer duration than 1 hour is converted proportionally to the duration of its appearance, which is incorrect.

Table 2

Normal Annual Precipitation in Poland for the Period Between 1891 and 1930

No.	Name of precipitation station	Elevation above sea level	Precipitation in mm	No.	Name of precipitation station	Elevation above sea level	Precipitation in mm
1	Pakość	75	443	20	Grodzisk	85	544
2	Włocławek	65	461	21	Siedlce	150	546
3	Nakło	70	471	22	Łowicz	97	547
4	Brześć Kujawski	110	472	23	Biała Podlaska	145	550
5	Płock	95	473	24	Lublin	196	551
6	Śrem	65	491	25	Gdańsk-Wrzeszcz	12	555
7	Gdańsk-Wodociągi	2	495	26	Szczecin	20	561
8	Toruń	34	495	27	Mikołajki	120	565
9	Modlin	107	500	28	Dęblin	117	566
10	Pułtusk	79	511	29	Brzeg Dolny	110	568
11	Poznań	60	517	30	Brześć on Bug	139	578
12	Warsaw-Mokotów	111	518	31	Chełm	188	579
13	Warsaw-Muzeum	127	550	32	Puławy	140	580
14	Bydgoszcz	46	522	33	Międzyzdroje	8	581
15	Białystok	136	522	34	Olsztyn	128	585
16	Hel	6	528	35	Białowieża	172	585
17	Żyrardów	116	535	36	Kołobrzeg	2	587
18	Tarnobrzeg	173	538	37	Radom	168	590
19	Kalisz	109	541	38	Wrocław	118	592

39	Wejherowo	30	592	64	Wieliczka	242	730
40	Łódź	208	604	65	Koszalin	41	737
41	Wieluń	196	604	66	Tarnów	204	739
42	Złotoryja	225	615	67	Kudowa-Zdrój	388	746
43	Łeba	3	619	68	Iwonicz	304	785
44	Zamość	216	629	69	Sanok	314	813
45	Otmuchów	210	636	70	Rabka	478	821
46	Zielona Góra	149	636	71	Krynica	599	853
47	Cracow-Rakowiec	216	636	72	Nowy Targ	593	864
48	Rzeszów	214	642	73	Porąbka	310	965
49	Suwałki	178	649	74	Cieszyn	300	966
50	Kielce	276	653	75	Poronin	743	975
51	Jarosław	204	662	76	Duszniki-Zdrój	556	996
52	Jelenia Góra	347	678	77	Karpacz	605	997
53	Częstochowa	259	678	78	Zakopane	833	1122
54	Dębica	215	685	79	Szklarska Poręba	640	1141
55	Koźle	172	691	80	Kościelisko	936	1148
56	Zabrze	256	698	81	Śnieżka	1602	1158
57	Cieplice-Zdrój	345	699	82	Wisła	433	1191
58	Katowice	264	701	83	Rycerska Górna	570	1309
59	Słupsk	20	701	84	Orle	825	1430
60	Przeworsk	203	706	85	Polana Chochołowska	1148	1500
61	Czorsztyn	500	709	86	Śnieżne Jamy	1490	1512
62	Przemyśl	204	709	87	Hala Gąsienicowa	1520	1715
63	Nowy Sącz	290	719	88	Morskie Oko	1393	1810

Another group of investigators solves this problem by proposing a scale for denoting downpours in correlation with the amount of precipitation and

Table 3

Berg Scale for Downpours

Duration of precipitation	Precipitation in mm	Intensity of precipitation in mm/min
5 min	2.5	0.50
10 ,,	3.8	0.38
15 ,,	5.0	0.33
20 ,,	6.0	0.30
25 ,,	7.0	0.28
30 ,,	8.0	0.27
40 ,,	9.6	0.24
50 ,,	11.0	0.22
60 ,,	12.0	0.20
2 hours	18.0	0.15
4 ,,	27.0	0.11
6 ,,	33.0	0.09
12 ,,	45.0	0.06
18 ,,	54.0	0.05
24 ,,	60.0	0.04

its duration. In 1905, the Berg scale for defining downpours was elaborated in Russia where it has been in use to the present time (Table 3).

Table 4

Highest Precipitation Quantities Observed in Poland

Observation station	Duration of precipitation	Precipitation in mm	Precipitation intensity in mm/min
Legionowo	1 min	8.1	8.10
Nowy Bieruń	5 min	25.3	5.06
Kopaniec	10 min	47.8	4.78
Walentynowo	30 min	90.7	3.02
Giełczyce	60 min	120.0	2.00
Zwierzyniec	100 min	134.0	1.34
Sienno	405 min	210.0	0.52
Witów	1 day	285.0	0.20
Hala Gąsienicowa	2 days	392.8	0.14
,, ,,	3 days	422.4	0.10

Another classification divides rains of high intensity into downpours and torrential rains. This was adopted by Chomicz, who published in 1951 a work which included classification of precipitation of high intensity. In addition to dividing precipitation into downpours and torrential rains, Chomicz divided each of these groups into several grades and prepared a diagram, in which a collection of parabolic curves divides the entire area of precipitation (between the axes of coordinates) into sectors of various shapes (Fig. 4). This diagram is of theoretical importance, while the Berg scale used in the Soviet Union is more convenient for practical purposes.

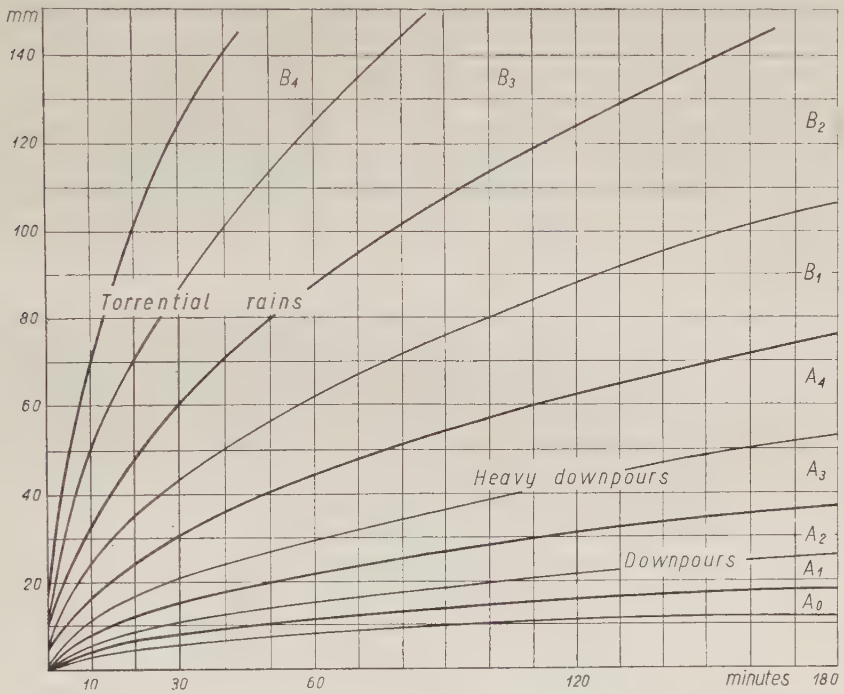

Fig. 4. Division of precipitation according to Chomicz

A collection of highest precipitations observed for some places in Poland is shown in Table 4.

Computing Precipitation Intensity

A hyperbolical correlation between the precipitation intensity i (in mm/min) and precipitation duration t (in min) may be expressed by the following formulas:

$$i = \frac{\Delta}{a + bt}$$

$$i = \frac{\Delta}{(c + t)^x}$$

where:

Δ — numerical parameter uniting the values i and t and sometimes called the strength of rain; the value of this coefficient is assumed in practice as identical for rains of the same frequency with the application of various possible correlations of i and t for identical frequencies;

a, b, c, x — coefficients.

In practice, however, simplified formulas are used, the parabolical correlation being very convenient.

Gorbachov recommends computing rain intensity by the follwing formula[2]:

$$i = \frac{\Delta}{t^{0.5}}$$

He affirmed that results very close to the truth are yielded by this formula. He elaborated a table of values Δ in correlation with types of rain (Table 5).

Table 5

Values Δ According to Gorbachov

No.	Kind of a rain	Δ
1	Small rains which do not cause runoff	up to — 1.0
2	Ordinary rains; runoff along cobbled street	1.1÷ 3.0
3	Large rains; runoff along natural slopes	3.1÷ 5.0
4	Ordinary downpours; great streams	5.1÷ 7.0
5	Heavy downpours; flooding of streets in towns	7.1÷ 9.0
6	Very heavy downpours; floods on small rivers	9.1÷12.0
7	Mountain downpours; floods in mountains	12.1÷16.0
8	Very large floods	16.1÷35.0

A table of correlation between rain intensity and duration of rain has been prepared by the Central Observatory in Leningrad. This (hyperbolical) correlation is shown in Figure 5 from which it is seen that the rain intensity rapidly decreases together with the increase in the duration.

Deriving formulas for the computation of the intensity of precipitation in

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[2]) Ogievskii: Gidrologiya sushi (Continental Hydrology). Moskva 1951, p. 188.

Poland has been attempted by many authors. Lindley was the first among them to prepare the formula for the city of Łódź in 1911, while two formulas for Warsaw

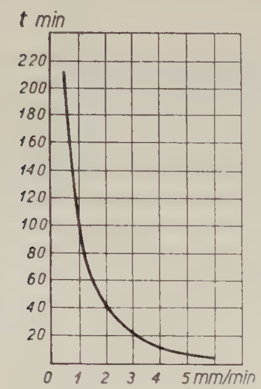


Fig. 5. Correlation between the intensity of precipitation and its duration

Table 6

Values of Coefficients for Computing Precipitation Intensity Established by Rożański

Region	Coefficients	
	a	b
Silesia	-0.187	4.829
Małopolska Region	-0.365	5.143
Wielkopolska Region	-0.400	5.576
Pomerania	-0.512	5.640

for various frequencies of precipitation appearance were worked up by Pomianowski. Rożański presented coefficients for the formula for computing intensity of precipitation in different regions of Poland derived on the basis of observations taken during the period between 1888 and 1910. However, the insufficient number of observations produced insufficiently accurate coefficients (Table 6).

Rożański used the following general formula derived by Hellmann:

$$i = a + \frac{b}{\sqrt[3]{t}}$$

where:

i — precipitation intensity in mm/min

t — rain duration in minutes,

a, b — coefficients.

In 1951 the following formulas for computing precipitation intensity were presented by a prominent Polish hydrologist, professor Rosłoński:

$$i = \frac{3.68004}{t^{0.5064}} \text{ for Warsaw}$$

$$i = \frac{3.61284}{t^{0.5}} \text{ for Cracow}$$

The precipitations computed by the Rosłoński formulas appear once in 5 years.

In 1952, there appeared a work by J. Lambor[3]) in which he indicated the following correlation between the intensity and duration of precipitation on Polish lands.

$$i = 17.164\,(t + 3.5)^{-0.39} - 1.44$$

The formulas presented can be used for computing intensity of downpours — i. e., rains of short duration and high intensity.

Lambor presents also the following formula for computing prolonged rains with high intensity, which he calls "spilled" rains:

$$i_{max} = \frac{534.912}{t + 1005}$$

Intensity i in these formulas should be expressed in mm/min and duration of rains t — in minutes.

The recently published works by Chomicz also discuss downpours on the territory of Poland. In these works, Chomicz recommends for computing precipitation intensity the use of the following simplified formula already referred to:

$$i = \frac{a}{t^{0.5}}$$

The contention of Chomicz who, together with the simple form of the formula and the exponent 0.5 also recommends the determination of the value of the coefficient to a 0.1 degree of accuracy should be considered correct. Inasmuch as measurements are often inaccurate and the numbers of observations usually insufficient deriving very complicated formulas or establishing with exaggerated accuracy individual numerals in formulas is not justified. Therefore although individual coefficients are determined with an accuracy of four or five decimal symbols the results obtained are only very approximate.

It has been proved by numerous investigations that the index of 0.5 quite adequately characterizes the course of the phenomenon. This was indirectly confirmed by Rosłoński, who in his formulas for computing precipitation intensity assumes for Cracow an index of power 0.5 and very close to that for Warsaw.

Computation of the precipitation intensity is linked with accurate measurements made of precipitation performed by pluviometers. It has been stated that, during strong winds usually accompanying downpours, the pluviometers now in use record diminished quantities of precipitation and, therefore, at wind velocities of, say 9, 12, 16 m/sec, the quantities of precipitation observed should be multiplied by 1.5, 2 and 3 respectively in order to obtain the real magnitude

[3]) J. Lambor: Związki charakteryzujące największe opady na ziemiach polskich (Correlations Characterizing the Highest Precipitations on Polish Lands). Hydrological and Meteorological Review. Vols. 1—2. Warszawa 1952.

of precipitation. The height at which pluviometers are placed also has an influence on the measurement results. At a smaller height, more precipitation usually gets into the pluviometer than at a greater height.

The layer of snow in a pluviometer does not usually correspond with its real thickness, a part of it being as a rule blown off by wind. The average error amounts to about 25 percent.

If the results of older observations are used, it should be remembered that they were taken with pluviometers of other heights and types than those at present applied yielding precipitation values lower than actual.

It may be assumed that a properly located pluviometer rain gage in the care of an experienced observer involves a measurement error of $\pm$ 10 percent.

The Course of Downpours

Heavy rains mostly appear over small areas, and rain of higher intensity usually covers smaller surfaces. It happens sometimes, however, that a very intensive rain appears over a large area.

Table 7 concerning surfaces on which appear precipitations of various intensities relates to the territory of the South Ukraine because no original table for Polish terrains is available. But the values presented would probably not deviate to any considerable extent from those valid for this country.

Table 7

The Occurrence of Various Intensity of Precipitation

Mean intensity of precipitation in mm/min	2.2–2	2–1	1–0.9	0.9–0.8	0.8–0.5	0.5–0.11
Area covered by precipitation in sq km	4–8	8–25	25–50	50–65	65–350	350–3500

A correlation of a hyperbolical character exists between the mean magnitude of the precipitation layer and the area on which it falls.

In an area covered by a downpour, the precipitation occurs very unevenly in separate elongated spots. The intensity of precipitation is not the same throughout such an area, being greater towards its center.

Downpours of short duration cannot be expressed in a given terrain by a continuous function which represents the course of changes in temperature, evaporation, atmospheric pressure, etc. The conversion of recordings of precip-

itation stations in order to obtain the total quantity of water which has fallen in a given area during a certain period is greatly complicated by the above circumstance.

When a period, for which it is intended to compute the total quantities of precipitation, is shortened such quantities will be very unevenly distributed over the area under investigation and, therefore, more observation points should be available for the determination of the real quantities of precipitation.

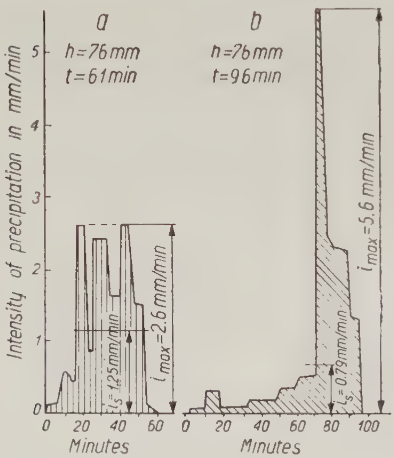


Fig. 6. Diagrams of the course of rains according to Dolgov

Two diagrams (Fig. 6) of rainfall yielding 76 mm precipitation observed by Dolgov by means of two self-recording pluviometers (pluviographs) are given as an example.

Figure 6a shows that the intensive rainfall started suddenly and stopped equally suddenly after 61 minutes. The highest intensity of the rain i_{max} reached 2.6 mm/min, while the mean intensity i_0 amounted to 1.25 mm/min. Another pluviograph, placed 4 km away showed, that the same quantity — i. e. 76 mm — was precipitated in that area by the same rain, although the course of the rain was entirely different. (Fig. 6b). At first for about 70 minutes the intensity of rain was low, and then suddenly increased. The rain, which lasted 96 minutes ended almost instantaneously. The highest intensity of the rain reached a value of $i_{max} = 5.6$ mm/min and the mean intensity amounted to $i_0 = 0.79$ mm/min.

Rains of great intensity mostly appear in Poland in the period between mid-May and mid-September. The period is somewhat longer only in Silesia.

Determining Quantities of Precipitation

To compute the maximum discharges, the total quantity of annual precipitations averaged from several years should be determined for a given catchment basin; this as already indicated is called the precipitation standard. Observations lasting over 50 and even 100 years are necessary to determine the real precipitation standard, because the mean precipitation values of shorter periods, — e. g. 20 years — are not stable and depend on the hydrological regime of individual observation periods.

Since it is not necessary for our practical purposes to determine permanent and accurate precipitation standards in a catchment area, we usually apply a 20 year or even shorter period, observation materials appropriate to a longer period

being available only for certain stations. For this reason, it would be more proper to term the annual precipitations averaged from a short period the arithmetic mean precipitation of such period, and not the precipitation standard.

Meteorological yearbooks serve for computing the total precipitation in particular years, while the data of the years for which yearbooks have not yet been published are taken directly from observation materials in possession of the Polish Hydro-Meteorological Institute.

The methods of computing total annual precipitation in a given catchment area are:

(1) the method of the arithmetic mean,
(2) the method of squares,
(3) the method of weights or polygons of uniform rainfall,
(4) the method of isohyets.

The Method of the Arithmetic Mean

An approximate quantity of precipitation in a catchment area may be determined by marking on the map the observation results of all the precipitation stations located in such catchment area, and computing the arithmetic mean of these results.

This method is admissible provided that:

(a) precipitation stations are fairly uniformly distributed in a catchment area;
(b) difference in observations of the adjoining stations does not exceed 10 percent;
(c) there are no localities with entirely different physiogeographical regime within the limits of a catchment area under study.

The non uniformity of distribution of the observation stations is overcome by adding "fictitious" stations which are marked on the map of a catchment basin in areas lacking such stations. The quantity of precipitation for these stations is determined by interpolation between the nearest stations, whereas it is assumed that a change in the precipitation between such stations takes place along a straight line — which is correct for flat lowland territories.

The Method of Squares

The method is simple and gives fairly accurate results. It consists in the division of a catchment area under study into squares of equal area (e. g. 625 sq km). A layer of precipitation for each square is subsequently computed as a mean arithmetic value of observations recorded by precipitation stations located in such square, or by interpolation between the adjoining observation stations. Values thus obtained are adopted as valid for median points of each square, and are entered onto each square.

The mean quantity of precipitation for the entire catchment basin is computed by summing up precipitations thus obtained for each square, and dividing this sum by the total number of squares. To check the results, the computation should be repeated for another distribution and different number of squares. Errors should not exceed 5 percent.

The Method of Weights or Polygons of Uniform Rainfall

The following procedure is used in application of this method. The existing precipitation stations 1, 2, 3 ... n are marked on the map of a catchment basin together with the precipitation values $H_1, H_2 \dots H_n$. These stations are connected

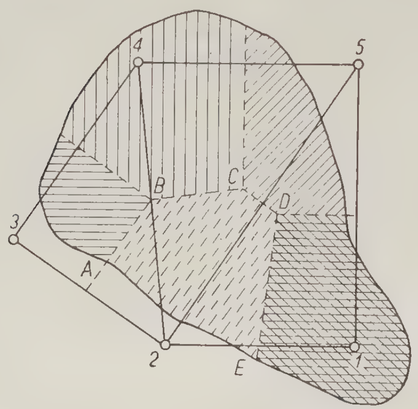


Fig. 7. Diagram of the polygons of uniform precipitation

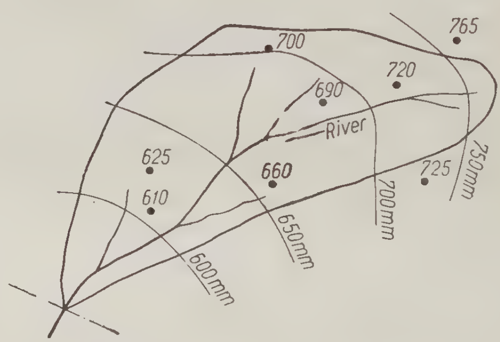


Fig. 8. Plan of a catchment basin with isohyets indicated

by straight lines so that a network of triangles is formed. Perpendicular lines drawn from the middle of each side of each triangle, bisect one another to form figures called polygons of uniform rainfall (Fig. 7).

The surfaces $F_1, F_2, F_3, \dots F_n$ of a polygon appropriate to each station should be established and, subsequently, the ratio of these surfaces to the surface of an entire catchment basin F should be computed. The weights for each station $W_1, W_2, W_3 \dots W_n$ are thus obtained.

Multiplying the precipitations $H_1, H_2, H_3 \dots H_n$, by the weight determined for each station, and summing up the products, the mean annual precipitation is obtained for the entire catchment basin (Table 8).

The Method of Isohyets

This method is considered to be the most accurate for determining the mean quantity of precipitation. Using this method, the annual totals of precipitation

Table 8

Computing Mean Precipitation by the Method of Weights

Station number	Precipitation on the station	Surface of polygons	Weight	Precipitation on the area of a polygon
1	H_1	F_1	$F_1 : F = W_1$	H_1W_1
2	H_2	F_2	$F_2 : F = W_2$	H_2W_2
3	H_3	F_3	$F_3 : F = W_3$	H_3W_3
.	.	.	.	.
.	.	.	.	.
.	.	.	.	.
n	H_n	F_n	$F_n : F = W_n$	H_nW_n

Mean annual precipitation in a catchment basin $\Sigma H_{1 \dots n} \; W_{1 \dots n}$

from all observation stations existing in a catchment basin during the period under study are marked on an appropriate map.

The isohyets, or lines of equal quantity of precipitation, are subsequently drawn in a manner resembling the tracing of contour lines on topographic maps.

The interpolation between observations conducted at the particular stations is made on the assumption of a uniform change in precipitation if the surface is of a lowland terrain. In mountain terrain, on the other hand, the topographical conditions should be taken into account by drawing isohyets denser on the slopes than on the plains.

Isohyets are usually traced through the points at which precipitation quantities differ by even numbers, e. g. 10 mm, 50 mm, 100 mm, etc. (Fig. 8).

The areas f_i, located between the isohyets are computed by means of a planimeter and also the mean arithmetic quantities of precipitation h_i are determined between the isohyets.

Multiplying particular values of areas and precipitations gives the sum $\sum f_i \, h_i$. Dividing this sum by the magnitude of the catchment area $F = f_1 + + f_2 \dots + f_n$, the value of a mean precipitation H in a catchment basin is arrived at.

$$H = \frac{\sum f_i h_i}{\sum f_i}$$

It is particularly convenient to express all values in meters, and to convert the final result into millimeters.

Snow

To compute the discharge during the spring rises, the quantity of water in the snow before it begins to thaw should be determined.

If a winter is characterized by abundant snowfalls, and a considerable layer of snow lasts until spring, then the influence of snow on the spring discharges may be considerable. The annual quantity of snow on Polish lands can reach 20 percent of an annual total of all precipitation.

The thickness of a snow cover depends on relief of the terrain, density of the vegetal cover, quantity of fences, hedges, snow-fences, etc. Strong winds can blow the snow off the open spaces, especially when there is poor vegetal cover. The blown snow fills the terrain depressions, and accumulates on the edges of neighboring forests (Fig. 9).

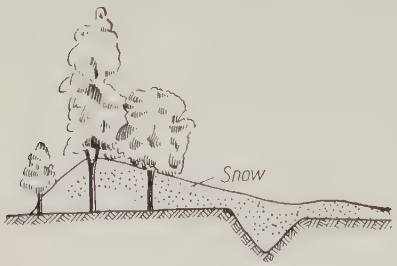


Fig. 9. Accumulation of snow near trees

The snow lies longer in places, protected against the action of sun and in forests which delay thawing, than in other terrains.

The snow thaws as a result of rises in air temperature, occurrence of warm winds or rains, or under the influence of sunrays directed onto its surface, especially when such surface is dirty. Snow usually begins to thaw when the mean daily temperature of air rises above 0°C, although a thaw can also happen under the influence of sunray action at a temperature below zero, even at -10°C.

It has been observed that solar radiation can penetrate the snow layer and reach the ground below it, therefore, in the spring snow may melt at its lower layers. This phenomenon appears very distinctly in thin snow layers. Moreover, the intensity of thawing increases as a result of water formed by superficial thaw seeping through the snow layer.

A decrease in the porosity of snow is caused by melting. Porosity may also be decreased during the thaw period, as well as together with the increase in the thickness of the snow cover resulting from the pressure exerted by upper layers of snow on the lower layers. Consequently, thickness of the snow layer cannot indicate the water contained in it and the extent of the snow porosity should additionally be fixed.

The height of the water layer h_w contained in the snow is computed from the following formula:

$$h_w = \frac{hG}{W}$$

where:

h_w — height of water layer contained in snow in mm,
h — numerical value of the height of the snow layer in mm,
G — numerical value of the weight of snow in grams,
W — numerical value of the volume of the same quantity of snow in cu cm.

Fresh fallen snow contains about 10 percent of water. The water content gradually increases to 25 percent as a result of the increasing thickness of a layer following the winter snowfalls, and reaches 40 percent before the beginning of thaw.

The intensity of the thawing of snow can be computed from the following formula:

$$a = 0.24\, t \sqrt{v + 0.3}$$

where:

a — intensity of thawing of snow in mm/hr,
v — wind velocity in m/sec,
t — air temperature in C grades.

Sokolovskii recommends adoption of the following intensity of the thawing of snow:

$$a = 7.5 \text{ mm/hr}$$

The thickness of the snow layer does not itself present a clear picture as regards the magnitude of spring rises appearing as a result of snow melting. Thus, for instance, a thick snow layer may cause no great discharge if the snow thaws gradually. Such a phenomenon may be observed, for instance, when the temperature rises above zero only in the day time, and ground frosts appear at nights, as often occurs in fact. On the other hand, the brief but high rises may be caused by much thinner snow layers if a thaw comes suddenly and temperatures above 0°C are recorded regularly throughout the entire thaw period.

The intensity of melting of snow is increased if a rain falls simultaneously. Although the rain alone can melt only insignificant quantities of snow, it exerts a considerable influence on the process of melting, because the raindrops falling down crumble the snow so that it melts quicker.

The quantity of the spring precipitation is usually several times lower than that in summer; even so, considerable rises may be caused by spring rains combined with water originating in the melting of snow, because during this period the soil is usually frozen and water cannot seep through the ground. Besides, the magnitude of evaporation is at this time low. Therefore, there are cases in which the maximum unit discharges formed by melted snow together with spring rains are not lower than the discharges caused by summer rains, irrespective of the size of catchment area.

3. Water Evaporation

Air Humidity

The transformation of water into vapor is called evaporation. The magnitude of evaporation is measured in millimeters of evaporated water layer.

Various quantities of vapor may be contained in the air or, in other words, the air may have various degrees of humidity. The quantity of moisture which may be contained in the air depends on air temperature; warmer air can contain a higher quantity of water vapor.

The vapor contained in the air increases the atmospheric pressure by a magnitude equal to the pressure of vapor alone. The pressure of vapor really contained in the atmosphere at a given moment is called absolute humidity.

The state of full saturation of air with vapor occurs when air contains the maximum quantity of vapor possible at a given temperature. When air is in the state of full vapor saturation, the vapor pressure (absolute humidity) at a given temperature reaches its maximum.

The atmospheric vapor pressure is expressed in millibars, or in millimeters of the mercury column.

The weight of vapor in grams contained in 1 cu m of air is approximately equal to the vapor pressure as expressed in millimeters. For this reason, the absolute humidity denotes also the quantity of vapor in grams contained in 1 cu m of air.

The degree of vapor saturation of air can be characterized by the relative humidity, or by saturation deficit also called underhumidity.

The relative humidity R is a ratio of the vapor content in air e to the quantity of vapor E which can be contained in air in a state of saturation at the same temperature. Relative humidity is practically always presented as a percentage, namely:

$$R = \frac{e}{E} 100$$

Air is fully saturated with vapor if the relative humidity amounts to 100 percent; if, for instance, it equals 50 percent, then air contains only 50 percent of that quantity of vapor which it would contain at the moment of full saturation.

The saturation deficit d is the difference between vapor pressure E at a given temperature and with full saturation and a real pressure of vapor e at the same temperature; or, in other words, it is the difference between the quantity of vapor necessary for the full saturation of air, and the quantity of vapor contained in it at a certain moment at the same temperature:

$$d = E - e$$

The magnitude of the saturation deficit is included as a principal element in certain formulas for determining evaporation, discharge, etc. To compute this value, the absolute humidity e should be known as well as that air temperature t for which vapor pressure E with full saturation of air is determined from psychrometric tables.

In order to establish the mean magnitude of the saturation deficit, such

deficit should be computed separately for all temperature observations conducted during the day, and subsequently for the entire day, month, etc. Such a computation is troublesome and tedious and, therefore, for computing the saturation deficit practice adopts the monthly mean air temperature and absolute humidity irrespective of the individual temperatures observed.

The correlation between the maximum vapor pressure and the temperature is not rectilinear and, therefore, the magnitude of the saturation deficit thus computed is somewhat lower than the real deficit. For this reason, to obtain the real value, a correction should be introduced, which is determined by the formula presented by the Russian scientist Oldekop:

$$\Delta d = 0.09\, A^2 \frac{d^2E}{dt^2}$$

where:

A — difference between the maximum temperature during a specific month and the mean monthly temperature,

$\frac{d^2E}{dt^2}$ — intensity of the change of vapor pressure under the temperature change taken from Table 9, as presented by Oldekop.

Thus, the formula

$$\Delta D = d + 0.09\, A^2 \frac{d^2E}{dt^2}$$

where $d = E - e$ denotes the saturation deficit in millimeters computed with the use of the mean monthly values E and e — should be used for computing the mean value of the saturation deficit D from the mean monthly magnitudes of absolute humidity e and air temperature t.

The Course of Evaporation

Factors Affecting the Course of Evaporation

The course of evaporation depends on three principal factors:

(a) the difference between the real vapor pressure in the air and pressure of vapor saturating the space above the evaporating area;

(b) temperature;

(c) wind velocity.

Evaporation continues up to the moment when the vapor pressure in the air equals the pressure of vapor saturating the space above the water or soil surface (at a given temperature). The higher the air temperature, the larger the quantity of vapor necessary for its saturation; therefore, evaporation is greater at a higher than at a lower temperature.

There is also greater evaporation in dry than in humid air, because the pressure of vapor is lower in the dry air.

Table 9

Values of Corrections in Computing Saturation Deficit

Temperature t°	Maximum vapor pressure E in mm	$\frac{d^2E}{dt^2}$	Temperature t°	Maximum vapor pressure E in mm	$\frac{d^2E}{dt^2}$	Temperature t°	Maximum vapor pressure E in mm	$\frac{d^2E}{dt^2}$
35	42,188	—	17	14,533	0.051	−1	4,255	0.021
34	39,911	—	16	13,637	0.049	−2	3,952	0.020
33	37,741	0.103	15	12,790	0.047	−3	3,669	0.019
32	35,674	0.099	14	11,989	0.044	−4	3,404	0.018
31	33,706	0.095	13	11,233	0.042	−5	3,158	0.016
30	31,834	0.091	12	10,519	0.040	−6	2,928	0.014
29	30,052	0.087	11	9,845	0.038	−7	2,712	0.014
28	28,358	0.084	10	9,210	0.037	−8	2,509	0.013
27	26,747	0.080	9	8,610	0.034	−9	2,321	0.013
26	25,217	0.077	8	8,046	0.033	−10	2,144	0.012
25	23,763	0.074	7	7,514	0.031	−11	1,979	0.012
24	22,383	0.071	6	7,014	0.030	−12	1,826	0.011
23	21,074	0.068	5	6,543	0.028	−13	1,684	0.010
22	19,832	0.064	4	6,101	0.027	−14	1,551	0.009
21	18,655	0.061	3	5,685	0.025	−15	1,429	—
20	17,539	0.059	2	5,294	0.023	−16	1,315	—
19	16,481	0.056	1	4,926	0.022			
18	15,480	0.054	0	4,579	0.022			

The rate of evaporation increases together with increase in the strength of wind, because wind in carrying moisture off the evaporating area delays the moment of saturating the air.

However, at wind velocity exceeding 5 m/sec., the magnitude of evaporation increases only slightly.

Loss of heat is caused by evaporation, because the transformation of one gram of water into vapor requires about 590 gramcalories of heat.

Explaining the phenomenon of evaporation and computing the quantity of water evaporating is of the utmost practical importance since the quantity of water flowing in streams depends to a considerable extent on evaporation.

In nature, evaporation takes place from the surface of soil by intermediation of plants and from the surfaces of water, snow and ice.

Evaporation from the Soil Surface

It is difficult to separate the evaporation from the soil surface from the evaporation taking place by intermediation of plants; moreover, for a hydrolo-

gist, such a discrimination is not usually necessary because he is interested rather in the total quantity of water evaporating in a catchment basin.

In the case of normal soil moisture, the evaporation from its surface is smaller than from the surface of water. An increase in soil moisture increases evaporation which in some cases is twice as high from the surface of thoroughly moist ground as from the surface of water. The diagrams of the intensity of evaporation from the surface of water and from the surface of a clayey soil are shown in Fig. 10.

Determining the magnitude of evaporation from the soil, it should be borne in mind that soil moisture not only changes under the influence of evaporating water and precipitation but also as a result of water movement inside the ground itself. Water movement inside the ground can take place downwards (infiltration) or upwards, when vapor rises from the lower layers of the ground. Water movements inside the ground are often disregarded and the evaporation from the soil is computed only on the basis of changes in the soil moisture, rainfalls being taken into account.

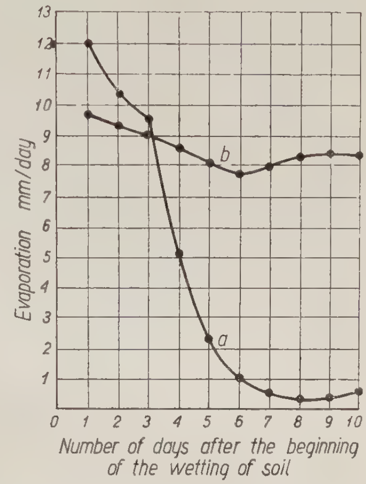


Fig. 10. Intensity of evaporation: *a* — evaporation from the surface of clay soil, *b* — evaporation from the surface of water

Table 10

Correlation Between Evaporation and the Color of Soil

Soil color	Magnitude of evaporation in %
White	100
Yellow	107
Brown	119
Gray	125
Black	132

Evaporation from the surface of south-facing slopes — better insolated and getting more heat — is 10-20 percent higher while evaporation from the surface of northern slopes is 10-15 percent lower as compared with the magnitude of evaporation from a level surface.

Evaporation from the soil surface increases together with the increase in the roughness and undulation of terrain, because the action of wind becomes stronger and the area of evaporation increases.

The intensity of evaporation decreases about 1.5 times following the loosening of soil and, consequently, the damaging of capillary orifices (capillary vessels)

through which water may ascend within the soil. Evaporation also depends on the color of the soil (Table 10).

Evaporation from Soil with Vegetal Cover

Evaporation from soil with vegetal cover includes:

(a) evaporation from the soil itself;

(b) evaporation of the precipitation retained on branches and leaves of plants;

(c) transpiration of plants.

Evaporation from the surface of soil covered with plants is much lower than from the surface of soil lacking vegetal cover.

Evaporation from soil between trees in a forest is about 2.5 times lower than evaporation from bare field soil; this is due to the higher humidity in a forest, weaker winds, lower soil temperature caused by shadows thrown by trees, and also to partial breaking of capillary vessels by the roots of trees.

It has been stated by Soviet investigators that soil is dried to the greatest extent under grass, somewhat less under grain, still less under forest, and least of all — in young groves. It has also been observed that soil moisture in forests increases after tree felling, although the upper layers of the soil become dryer.

A certain amount of precipitation is retained on leaves, branches and trunks of trees and evaporates in the same manner as, for instance, the water evaporating from the surface of soil or wet stones. Thus, about 39 percent of the annual precipitation is retained by coniferous forests, 13 percent by deciduous forests, and about 5 percent by grass.

Precipitation lower than 1 mm is entirely retained on trees, lower than 5 mm up to 70 percent, and high precipitation up to 24 percent.

Water retained on leaves and branches does not feed plants and does not change the degree of soil moisture. This phenomenon is, however, extremely important hydrologically because it speeds up water circulation and increases the quantity of rainfall.

Vegetal cover greatly influences evaporation from the soil surface. Transpiration takes place in connection with the life and development of plants, consisting in the phenomenon that each plant takes water from the soil through its roots and conveys it up to the surface of leaves. Only $\frac{1}{200}$ to $\frac{1}{700}$ part of water thus taken is used for the development of plant tissues; the residue (over 99.5 percent) evaporates from the surface of leaves or — in other words — is used for transpiration, which continues without interruption even when the air is saturated with moisture.

Transpiration of plants depends primarily on:

(a) type of vegetal cover;

(b) degree of its development and age;
(c) soil moisture;
(d) meteorological conditions (temperature, air humidity, wind, radiation, etc.).

The transpiration of plants is higher during the first half of the vegetative period than in the second. Transpiration is very low when plants begin to wither. If the moisture content of soil is low, plants adapt themselves and take smaller quantities of water.

A fairly great evaporation is caused by vegetal cover of marshes. According to Dubakh (USSR), the evaporation from the surface of moss overgrowing marshes can be 15-20 percent higher than from the water surface.

Evaporation from Water Surface

Computing the magnitude of evaporation from the surface of water involves measuring the water layer which has evaporated during a certain period. Although the technique of conducting such measurement is very simple, the available results of direct measurements of evaporation from water surface cannot usually be applied due to the inaccuracy of instruments used for this purpose.

This is the reason why the magnitude of evaporation from water surface is usually computed by empirical formulas which take into account saturation deficit and wind velocity.

The following Meyer formula is generally used for computing evaporation from water surface:

$$E = d\,(15 + 3\,w)$$

where:

E — layer of water evaporated during one month, in mm
d — mean monthly saturation deficit in mm (regardless of the Oldekop correction);
w — mean monthly wind velocity at a height of 10 m.

The Meyer formula was derived from observation of evaporation by evaporation gages (atmometers) of small diameter, which record excessive magnitudes as compared with the real course of evaporation. A reducing coefficient should be introduced to obtain more accurate results. Then the Meyer formula will have the following form:

$$E = Rd\,(15 + 3\,w)$$

where R denotes the reducing coefficient.

Davidov recommends that the reducing coefficients be subjected to the mean monthly saturation deficit (Table 11). Thus there is a possibility of establishing the magnitude of the reducing coefficient for each month separately in correlation with the real conditions of evaporation.

Table 11

Values of Reducing Coefficients R According to Davidov

d	15.0	10.0	8.0	6.0	5.0	4.0	3.0	2.0	1.5	1.0	0.5	0.3	0.2	0.1
R	0.52	0.57	0.60	0.65	0.70	0.76	0.84	0.95	1.00	1.09	1.20	1.25	1.30	1.40

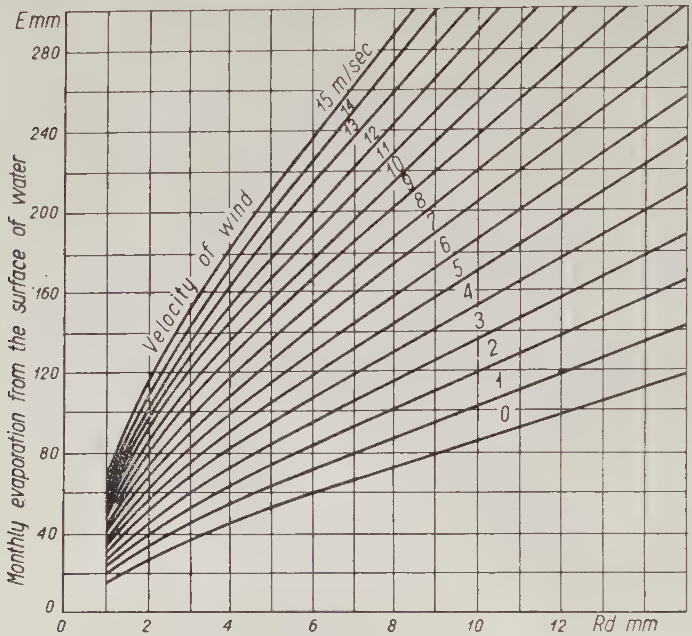


Fig. 11. Diagram for computing evaporation from the surface of water

A diagram, shown in Fig. 11 facilitates computations.

A simple formula presented by Moklak in 1940:

$$E = 30\, d^{0.7}$$

may serve for determining the approximate magnitude of evaporation.

Under Poland's climatic conditions, the average thickness of the layer evaporating from the surface of water during one year amounts to about 970 mm; 75 percent of that falls to the months between May and October, and 25 percent to the winter months — i. e., from November of the previous year to the following April. The highest evaporation from water surface during one month takes place in July and amounts to about 200 mm which yields an average of about 6.7 mm per day. The evaporation on very hot days may be much higher.

Evaporation from the surface of snow and ice has been insufficiently investigated thus far. Under normal conditions, the magnitude of such evaporation

is not high, although it also takes place during frosts (Table 12). Evaporation from ice surface is about half that from the snow surface.

Table 12

Snow Surface Evaporation

Temperature in degrees	Evaporation in mm per day	Temperature in degrees	Evaporation in mm per day	Temperature in degrees	Evaporation in mm/day
8	8.7	2	2.0	− 4	0.67
7	7.5	1	1.5	− 5	0.56
6	6.3	0	1.0	− 7	0.40
5	5.2	−1	0.9	−10	0.23
4	4.0	−2	0.8	−15	0.13
3	3.0	−3	0.7	−20	0.10

Computing the Entire Evaporation from a Catchment Basin

Particular types of evaporation from the surface of soil, water, ice and snow, as well as by the intermediation of plants, have already been studied. In hydrological computations, the total parameters of a catchment basin should in general be computed without dividing them into particular elements. Such a presentation of the problem simplifies solution.

An equation of the water balance can serve to compute the sum of evaporation from a catchment basin. Water participating in the water balance of a catchment basin reaches the area of a catchment basin in the form of:

(1) precipitation x_1 falling on the ground surface;

(2) water x_2 reaching the soil of a catchment area following the condensation of vapor in the atmosphere, together with water ascending from the deeper soil layers;

(3) subsoil flow x_3 from the adjoining catchment basins.

Loss of water on the surface of a catchment basin takes place as a result of:

(a) surface runoff h_1,

(b) evaporation from the surface of soil and water and transpiration z,

(c) subterranean runoff to the areas of the adjoining catchment basins h_2.

All the water resources existing in a catchment basin prior to the beginning of the period under study should also be taken into account in an equation of the water balance for the period studied. Let us denote the resources of surface waters (contained in snow, ice, plants, etc.) by k_1, and the resources of subsoil waters by p_1. Now, let us denote water resources which will be contained in a catchment basin at the end of the period under study, respectively by k_2 and p_2.

Then, the equation of the water balance in a catchment basin will have the following form:

$$x_1 + x_2 + x_3 + k_1 + p_1 = h_1 + z + h_2 + k_2 + p_2$$

Approximately, we may assume that:

(a) the quantity of water x_2, which is not recorded by pluviometers, is equal to 0;

(b) the quantity of ground water x_3, inflowing from the adjoining catchment basins is equal to the quantity of water h_2 running off by an underground route to the areas of the adjoining catchment basins, or $x_3 = h_2$;

(c) the resources of the surface waters k_1 at the beginning of a short period — e. g. a water year (hydrological year) are equal to the resources k_2 at the end of such period.

As to the resources of ground waters, their quantities at the beginning and end of a water year may vary considerably. Substantial stores of such waters may be accumulated in summers marked by heavy rains, because they have not time enough to reach a river as a result of the slowness of their movement. Rivers will be partially fed by this store in a dry summer. It follows from this that a store of these waters at the beginning and end of a water year may differ considerably, whereas the magnitude $(p_2 - p_1)$ may have different signs in particular years:

$$\pm (p_2 - p_1) = \pm \Delta p$$

After such assumptions and simplifications, the equation of the water balance shown above will have the following form:

$$x_1 = h_1 + z \pm \Delta p$$

This equation is called a simplified equation of the water balance in a catchment basin.

If equations of this type are prepared for observations of a long series of years, then it appears probable that after summing them up and deriving a mean value — the component $\pm \Delta p$ can be discarded.

Thus we obtain the correlation:

$$x_1 = h_1 - z$$

On the basis of this correlation, Meyer presents the method of computing the runoff as a difference between precipitation and evaporation:

$$x = h - z$$

where:

x — mean multiannual layer of runoff;

h — mean multiannual layer of precipitation;

z — mean multiannual layer of evaporation.

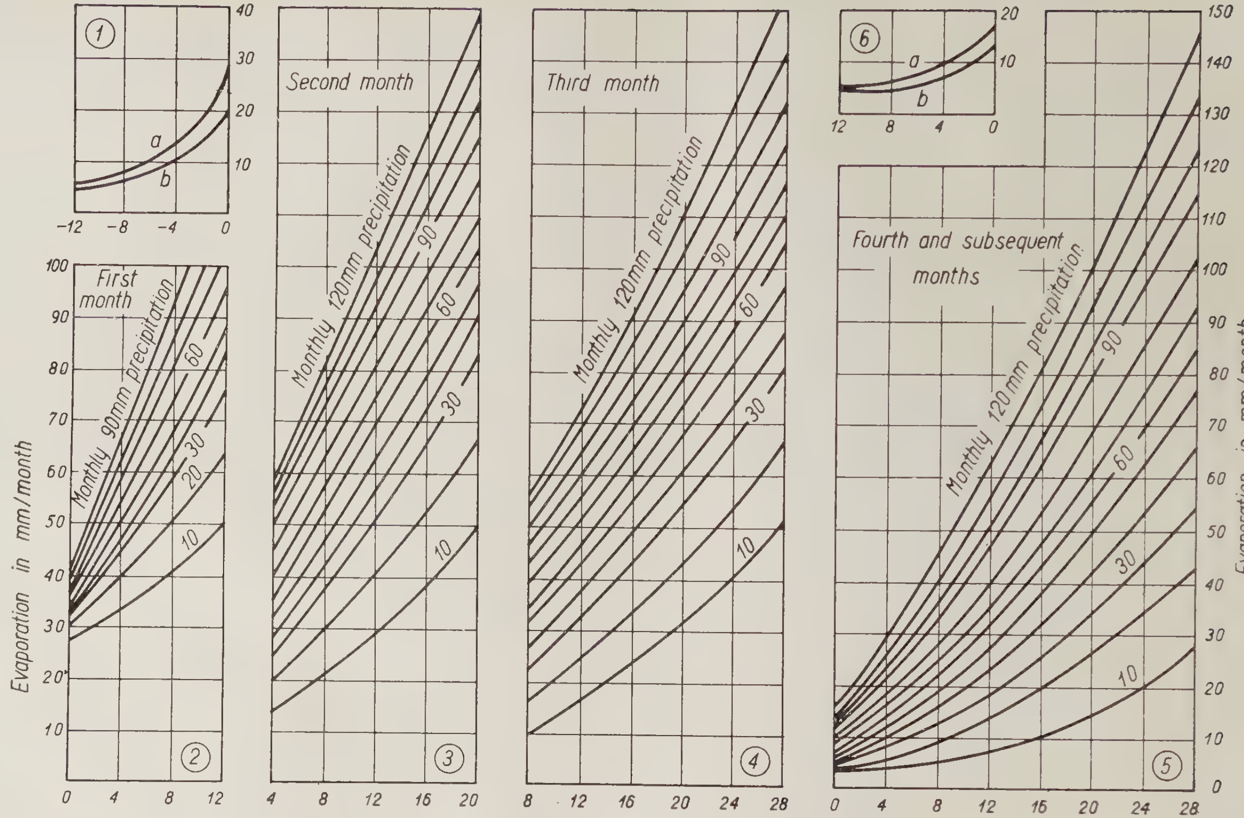


Fig. 12. Polyakov diagrams for computing full evaporation

4

The computation of the total evaporation z consists in summing up three types of evaporation:

(a) from the soil surface;

(b) from the water surface;

(c) by the intermediary of plants.

Meyer recommends computation of evaporation for each month separately, according to the mean air temperatures in particular months. Monthly layers thus obtained should be totalled to arrive at an annual evaporation.

Greater accuracy in computing the total evaporation in catchment basins is achieved by the Polyakov methods taking into account not only the temperature and precipitation, but also the ground moisture.

Six diagrams prepared by Polyakov, adapted to Polish conditions, are shown in Fig. 12. The computation is started with diagram 1, used to determine the evaporation in January, February, and the subsequent months with negative temperatures. For those months, as also for months with negative temperatures following the fall — for which evaporation is computed on the basis of diagram 6, — the magnitude of monthly precipitation is quite insignificant and may be disregarded, while the magnitude of evaporation is established exclusively from temperatures.

The curve denoted by letter a in diagrams 1 and 6 refers to areas within geographical latitude $\geqslant 52°$, while the curve denoted by the letter b is valid for areas within geographical latitude $< 52°$.

Evaporation in the first month of positive temperature should be computed using diagram 2, in the second — diagram 3, in the third — diagram 4, in the fourth, fifth and subsequent months — diagram 5.

In the event of the appearance of identical temperatures and precipitations, diagram 2 yields the highest evaporation, since after the snow has all thawed the ground moisture reaches its maximum, which contributes to the increase in evaporation.

Note that evaporation has a considerable influence on the magnitude of the annual or periodical flow in designing dams, ponds, irrigation canals, etc. Lesser influence is exerted by evaporation — particularly in the case of small rivers — on the maximum discharge per second, which operates in designing bridges and culverts.

4. Catchment Basin

Size and Shape of a Catchment Basin

Water falling on the surface of the ground accumulates in terrain depressions, and also runs off along the surface under the influence of the action of the force of gravity and following the direction of existing slopes.

The formation of surface runoff is related to a marked degree with the process of origin of ground waters, as well as with the movement of such and their getting to the surface of the ground.

It has been found on the basis of detailed investigations, that ground (subsoil) waters are formed by two factors — precipitation seeping through the ground and vapor condensation.

Atmospheric precipitation and sources of ground water do not immediately form great rivers. At first, water is accumulated in small streams from which larger ones and, subsequently, great rivers are developed. Exceptional are rivers originating from marshes and lakes which even in their upper reaches can be quite wide — as, for instance, the Neva River flowing from Lake Ladoga. Its breadth is practically unchanged from source to mouth.

Surface streams which — according to their size and existing physio-geographic conditions — may be continuously or temporarily filled with water, form a hydrographic network. Streams belonging to a hydrographic network, depending on the direction of slopes of the ground surface, feed the separate main water arteries flowing into seas and oceans.

An area from which a river is fed with water is called a catchment basin (or river basin). Catchment areas of the adjoining streams are divided by watersheds or ridges cutting through the highest points of the terrain (Fig. 13).

Fig. 13. Catchment basin diagram

The surface of a catchment basin constitutes its principal geometrical characteristic. The volume of water flowing through a catchment basin as well as formation of flow depend on the size of its area.

The area of a catchment basin is computed in square kilometers or, sometimes, in hectares (for reclamation purposes). In determining the catchment basin area, a distinction should be made between a geographical area and an area effective for water runoff. The geographical area covers an entire catchment basin, within the boundaries of which separate areas may be located closed by separate watersheds from which no endorheic drainage takes place. The difference

between these two kinds of area may be fairly large and, therefore, in hydrological computations particulary those pertaining to small rivers, the entire geographical area of a catchment basin cannot be taken into account, but should be reduced by the magnitude of areas of endorheic drainage.

Apart from the catchment basin on the surface of the ground, each river has its own subsurface basin hidden underground. Although the areas of these two basins may be different, in computations they are generally assumed to be identical, because the determination of the area of a subsurface basin is difficult and mostly of no considerable consequence to the discharge (Fig. 14).

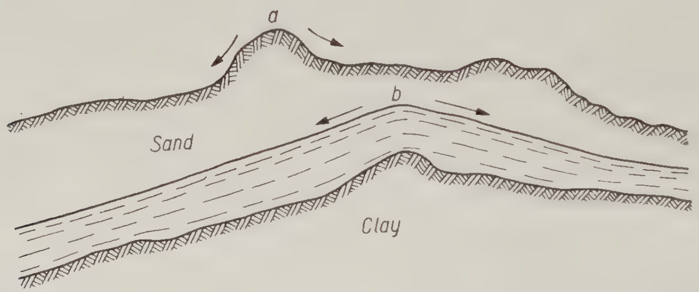


Fig. 14. Watershed diagram: *a* — topographic divide, *b* — ground water

The magnitude of the surface runoff can be greatly changed by high rises as a result of water spilling over the watershed line of two adjoining catchment basins. The water spill from one catchment basin to another is dangerous, particulary for small structures, because under favorable conditions it may even increase several dozen times the discharge.

The length of a river is its length from sources to mouth measured along the axis. The river mouth is usually adopted as zero point in measurements of river length since it is easier to fix than the point at which a river begins its course.

The length of a line drawn from the river mouth to the most distant point of a catchment basin is adopted as the length of such basin; this line is traced through the centers of transverse lines bisecting the area of a catchment basin (Fig. 15).

The width of a catchment basin is the ratio of the catchment basin area to its length.

The size of a catchment basin area gives a notion of the length of routes by which water runs off to rivers. Other conditions being equal the shorter the route of water runoff the smaller the water losses involved in evaporation and water infiltration through the soil.

The smaller the catchment basin the less uniform is the runoff during the entire year. Under identical climatic conditions, the duration of a spring rise

is shorter in a smaller catchment area; the beginning of a rise takes place somewhat earlier in small catchment basins than on large rivers.

The phenomenon of water drying up may appear on very small rivers in summer or freezing right to the very bottom in winter while on medium size rivers such phenomena may occur only exceptionally.

Usually, the discharge per second is also greater in larger catchment areas. Not only the size of a catchment area may exert considerable influence in this respect but also shape, length and slope.

The influence of the shape of a catchment basin arises from a shortened or lengthened route over which every separate water particle moves. The most favorable discharge conditions arise in catchment basins rounded and symmetrical in shape. The magnitude of discharge decreases with elongation of the shape of the catchment basin, because the inflow of water particles from the catchment basin takes place at different times.

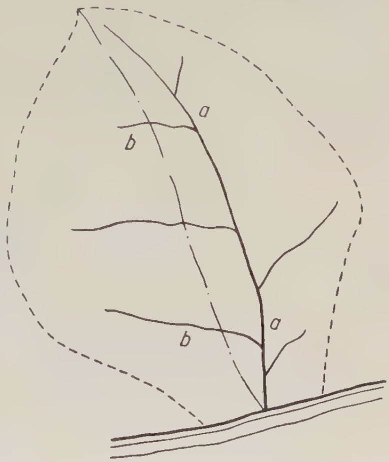


Fig. 15. Catchment basin diagram: *a* — river, *b* — length of a catchment basin

A rise occurs as water passes along the river and, therefore, primarily depends on its length. The width of a catchment basin, on the other hand, depends on changes in its length and, therefore, the intensity with which the main channel along the river is fed also depends on the length of the catchment basin.

The influence of the shape, length and slope of a catchment basin are disregarded by the majority of existing empirical formulas for computing maximum discharge per second; satisfactory results are not, therefore, to be obtained with these formulas. The flow in cu m/sec grows as a rule together with increase in the area of catchment basin, while the maximum discharge per second can be increased or diminished in correlation with the existing conditions of evaporation, seepage, draining for reclamation purposes, etc.

The Volga River is a classical example of the influence exerted on the maximum discharge per second by the length of the river and the ratio of its catchment basin to its length.

A diagram of peak discharges on a certain sector of the Volga during the 1926 rise, the highest in this century in its central and lower reaches, is shown in Figure 16.

That figure shows that, although the catchment basin increases in the sector between the confluences of the Oka and Kama Rivers from 479,000 sq km to

671,000 sq km — i. e. by 40 percent — the maximum discharge per second increases by only 1 percent, because the ratio of the mean catchment basin width B to its length L — is halved.

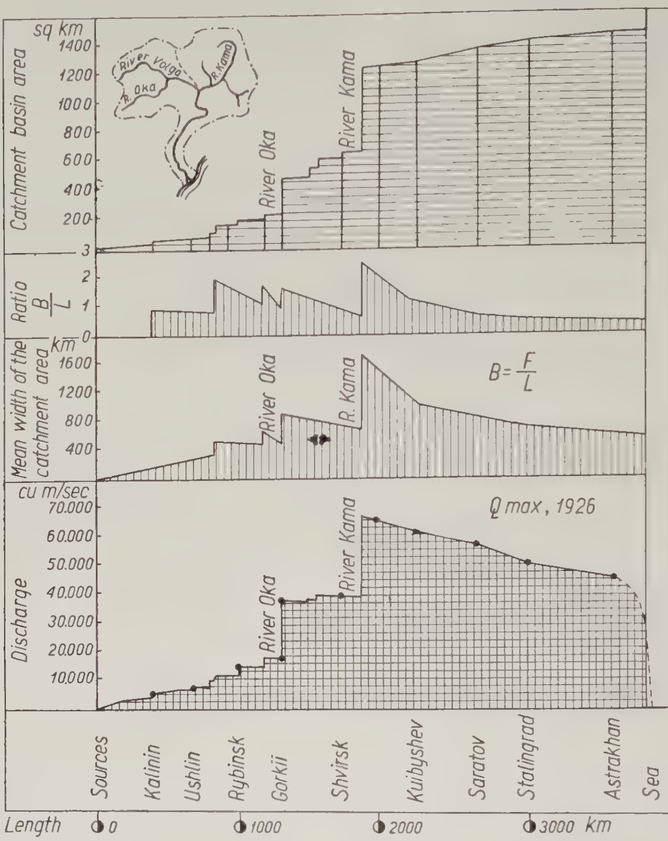


Fig. 16. Correlation between maximum discharges and catchment basin area, mean width of a catchment basin, and length of the River Volga

The area of the catchment basin F below the River Kama mouth is more than halved, and the discharge per second is simultaneously almost doubled because the ratio of the width to the length of the catchment basin was trebled.

The catchment basin increases slightly (by about 10 percent) between the mouths of the Kama River and the town of Astrakhan, while the discharge falls 35 percent because the ratio of the width to the length of the catchment basin is multiplied seven times. It should be noted, that, simultaneously, the annual discharge increases on this sector as a result of the catchment basin spreading out.

This example shows that the ratio of the width to the length of the catchment basin may greatly influence the maximum discharge.

Different maximum discharges may also appear with identical ratio of width to length of the catchment basin. For instance, the ratio of width to length of the catchment basin A shown in Figure 17 is identical with the ratio of these two magnitudes in an adjoining catchment basin B. Nevertheless, the discharge in the catchment basin B will be slightly higher than in the catchment basin A because the shape of the two catchment basins is decisive here.

The configuration of a catchment basin is usually characterized by the slopes of its surface and the streams flowing through its area on which mainly depends the magnitude of runoff. In other places, under identical conditions, water runs off quicker along a steeper surface and, therefore, the maximum discharge will be higher than in a catchment basin with a milder slope, while in such a catchment basin the duration of a rise will be shorter.

With a decrease in the duration of water flow together with an increase in slope, the losses involved in evaporation and infiltration are also lower. This causes a more intensive runoff in a mountain than in a lowland terrain. Moreover, the evaporation in a mountain terrain is still lower as a result of lower temperatures.

Note that a particularly high decrease in the maximum discharge per second appears in flat and lowland terrain. Factors principally responsible for the appearance of this phenomenon are the capacity to retain water in terrain depressions

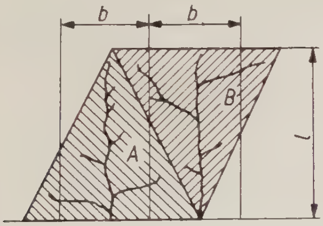


Fig. 17. Different shapes of two catchment basins of equal width

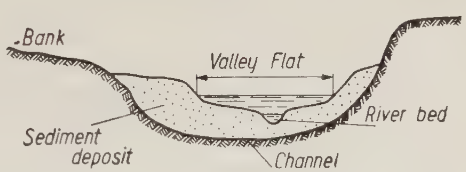


Fig. 18. Cross section of river valley

lacking effluence characteristic for such types of terrain, together with — inherent in such catchment basins — an insignificant water velocity in areas with a slight slope.

The formation of a unit runoff also depends on the character of a river channel. Note that a river channel with a low flooded bank and a wide valley flat exerts a dominant influence on discharge because large amounts of water are accumulated. The regulative influence of valley flat of marshy rivers on which spring and summer rises are mild is especially apparent.

A part of a river valley along which water flows is called a river channel,

whereas a river valley is a terrain depression filled with sediment, in which river water has carved its channel. An area flooded during highwater stages (Fig. 18) is called a valley flat.

The increase in water flow velocity is closely correlated with changes in the slope of the catchment basin slope i. Investigations have been shown that the velocity is proportional to $i^{0.25}$.

Unit runoff also depends on the elevation of the terrain. The height of a precipitation layer grows with increased elevation and greater magnitude of unit runoff may be expected in higher situated areas.

The magnitudes of precipitation and runoff in particular parts of the River Tissa (tributary of the Danube) catchment basin, divided into 5 sectors in correlation with the elevation are shown in Table 13. This Table shows that the flow coefficient is higher in the highest and most montainous part of the catchment basin and its value diminishes with decrease in elevation, to reach 0.22.

Table 13

Variation of Flow Coefficients

Name	Catchment basin category				
	I	II	III	IV	V
Precipitation x in mm	1174	795	732	710	631
Flows h in mm	600	253	202	196	138
Flow coefficient $h : x$	0.51	0.32	0.28	0.28	0.22

In high mountains, the flow coefficient can approach unity.

The higher value of flow coefficients for mountain rivers is caused by the appearance of greater precipitation in the catchment basins of these rivers as well as by steeper slopes.

The values of total flows for a period of many years depend to a much lesser extent on the terrain configuration than do flows caused by individual rains. If, during a spring rise, a higher maximum appears in a catchment basin with steep slopes than in an adjoining catchment basin with milder slopes, the total flows should probably vary only slightly provided that the losses involved in evaporation and infiltration are identical.

However, since the losses in a catchment basin with steep slopes should in principle be somewhat lower, some slight decrease in the total magnitude of efflux in flatter catchment basins will probably appear. There is no doubt that a considerable decrease in the total efflux during an annual or multiannual period will appear in very flat lowland catchment basins as a result of the specific properties of such basins referred to above.

The drainage density of the river network (Fig. 19) is usually expressed by the ratio of length (in km) of all streams $\sum l$ on the surface of a given area to the surface of such area P in sq km, or:

$$f = \frac{\sum l}{P}$$

Note, that the characteristics of a river network density thus arrived at can give only a relative idea concerning the degree of its development in two areas compared and that provided that the length of networks is determined from maps elaborated in identical scales. Small streams may be omitted from small scale maps; in that case, the length of a river network will be smaller than when computed from larger scale maps.

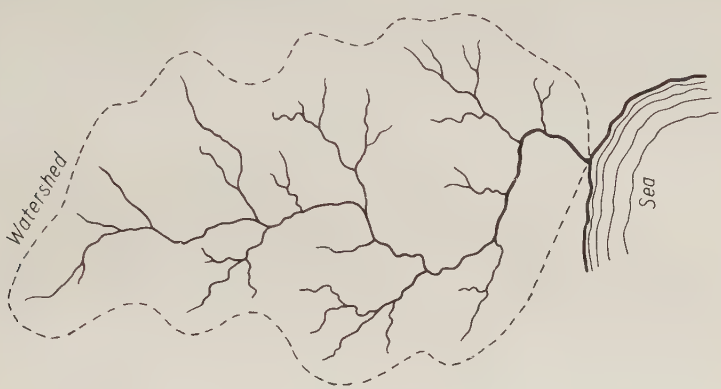


Fig. 19. Diagram of a river network

The drainage of a river network indicates the drainage capacity of a catchment basin which is dependent on the action of many factors connected with water movement and, at the same time, determines the magnitude and velocity of flow in a catchment basin.

The density of the river network depends on the type of soil in a catchment basin, terrain configuration, vegetal cover, quantity of precipitation, etc.

In permeable grounds, a major part of precipitation reaches river channels as a subsoil runoff and, therefore, in such areas the river network is less developed. The steeper the slope of the terrain, the denser the river network.

The density of a river network in mountain catchment basins is usually greater than in the lowland basins, because the former are marked by higher quantities of precipitation and less permeable grounds.

The density of the river network in afforested areas is slightly smaller than

in a terrain without woods. Afforested areas have more favorable conditions for water infiltration and evaporation.

In catchment areas of identical physio-geographical conditions but marked by a different degree of river network development, an increase may be observed in the magnitude of runoff if the density of a natural and artificial river network is increased.

The correlation of runoff with the degree of density of the canal network has been presented by the Soviet North-West Scientific Research Institute for Hydrotechnics and Reclamation in the following formula:

$$M = 0.24 \frac{f}{b^{0.28}}$$

where:

M — runoff in liters/sec on one sq km,

f — degree of network density, obtained as a quotient of canal lengths in km and size of the area in sq km,

b — percentage of terrain marshiness.

In an identical marshiness of terrain, the runoff is directly proportional to the degree of the density of the canal network.

Water Movement Within the Catchment Basin

Water flows in streams as a result of the operation of the form of gravity and, strictly speaking, of the component of the force of gravity directed parallel to the axis of a stream.

It is usually accepted, that:

(a) every water particle moves in a stream parallel to the main direction of the entire mass of water;

(b) water movement velocity at the bottom equals 0;

(c) maximum velocity usually appears just below the surface of a stream of water;

(d) the entire water movement depends on the adherence of particles and the resistance is proportional to the current velocity in the first power (the Darcy formula).

Yet, under normal conditions, a movement called regular or laminar (Latin "lamina" — a layer) is very rarely met with. It happens only when ground waters move very slowly within fine-grained grounds, or water runs off along the surface of the terrain up to the moment when it begins to form rivulets.

As the velocity increases, the stream passes at first into an unsteady stage and, subsequently, begins to flow with a turbulent movement (Latin "turbulentus" — disturbed) in which water particles constantly change their places. In the turbulent movement, resistances grow proportionally to the squares

of velocities (the Chezy rule). The velocity at which both kinds of movement change is called critical velocity.

The general direction of the entire water mass in a stream is in accord with the direction of the route taken by the river. When the water level rises above the channel banks and begins to fill the valley flat it shows a tendency to flow according to the direction of the river valley, and not according to the direction of its bed. Two streams are formed as a result of this tendency: the upper, broad one with a direction approaching that of the river valley and a lower narrow one, moving along the river bed (Fig. 20.).

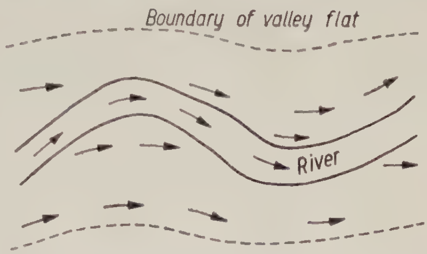


Fig. 20. Direction of the water movement in a main channel and in valley flats

The mutual reaction of these two streams is complex and so far has been little investigated. It may be supposed that large eddies and turbulences, which can change the shape of a channel, are formed in the planes of contact of both these streams.

The magnitude of maximum runoff in a river also depends on the direction of its tributaries. If the tributaries are radially located, the feeding of a main river takes place almost simultaneously, causing an increase in the flood wave

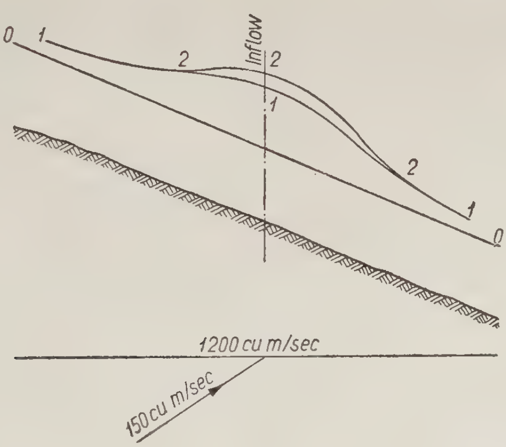


Fig. 21. Influence of a tributary on the magnitude of the flood wave: 0—0 — longitudinal cross section of the water level for average water stage, 1—1 — flood wave, 2—2 — increase in the flood wave caused by a tributary

(Fig. 21). In elongated catchment basins, on the other hand, the main river is fed gradually by tributaries, thus having a lesser influence on the formation of a rise.

In large catchment basins, the direction of the water movement in a river and the direction in which the water produced by snow thawing in a catchment basin runs off, exert a considerable influence on the character of a flood wave. A flattening of a flood wave appears on rivers flowing southwards. Under such circumstances, a considerable quantity of water produced by melted snow is absorbed by a main river and flows towards the river mouth before the arrival of a flood wave from the upper parts of a catchment basin.

The direction of the water movement in rivers flowing northwards is in accordance with the direction in which the snow thawing in a catchment basin takes place. In this event, a flood wave moving downstream is gradually fed with water produced in a catchment basin by snow thaws as the flood approaches the river mouth.

The Soil and Geological Structure of the Catchment Basin Ground

Ground retention, or the ability to retain rain water under the surface of the ground has considerable importance in the formation of runoff. This water subsequently flows by way of its underground channels to the river feeding it during the rainless period.

Permeability of the ground to facilitate water penetration between the soil particles is a necessary condition for the formation of water stores hidden within the ground.

Water can filter through the ground in places where it falls in the form of rain, as also on the terrain surface during the flow of the surface waters. Various types of resistance — such as friction, adherence, etc. are encountered by water particles in the ground.

A ground offering great resistance is called impermeable ground because the water particles inside can move only very slowly and only a very small quantity of water can filter through it in a unit of time. It is accepted in practice that no water at all is passed through impermeable ground.

Sandy grounds are marked by high permeability while clay is usually impermeable. For this reason, sands contribute to a decrease in surface flow and to an increase in underground flow. Clay behaves quite differently, since it neither decreases the surface flow nor causes the underground feeding of a river.

Permeability of ground depends on its chemical composition, geological structure, size of individual grains, homogeneity, shape of individual parts and manner of the distribution of such, in relation one to another; in other words — it depends on the ground structure.

The chemical and physical properties of ground are subjected to changes as well as its permeability and absorption.

Thus, for instance, ground highly saturated with water becomes impermeable and, therefore, increases the quantity of surface flow. Dry grounds, on the other

hand, absorb water very intensively decreasing surface flow. Frozen grounds become impermeable irrespective of structure.

The presence of a large quantity of earthworms in the soil of a catchment basin increases water infiltration, because, moving inside the ground, they create small underground passages and loosen the ground. Similar effects are caused by moles and other creatures, which, digging their burrows and other hiding-places in the soil, cause a decrease in the magnitude of the coefficient of surface flow.

The formation of runoff is notably influenced by the geological structure of the catchment basin. The occurrence of impermeable grounds in a catchment basin causes a sudden rise in water level in the event of downpours and thawing snow.

If cracked mountain layers exist in a catchment basin, a considerable part of the precipitation rapidly permeates inside the ground and again reaches the surface in the form of springs, or moves underground for a long time.

The geological cross section of ground with marked ground and artesian waters is shown in Figure 22.

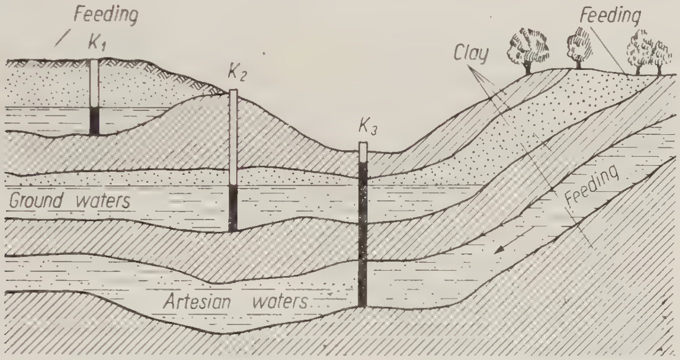


Fig. 22. Geological cross section of the soil

Water movement within the ground depends to a considerable extent on the dimensions of its inside openings, which might be divided into non capillary and capillary openings or pores.

Non capillary openings can appear in the form of crevices, channels and caves, through which quantities of subterranean water may flow similarly as in rivers. Small openings, width not exceeding 0.25 mm, are reckoned among capillary channels by means of which water is raised within the ground above the level of the ground waters.

The size of individual ground particles influences the dimensions of spaces left between them and, consequently the cross section of small channels, by means

of which water filters through the ground. Larger diameter of channels causes greater ground permeability since it facilitates the filtration of water through the soil in a shorter time.

Permeability is also influenced to a considerable extent by the shape of individual soil grains. In homogeneous grains, the largest empty spaces and highest diameters of channels appear in grains of spherical shape, and the smallest in cubic shapes. In non homogeneous grains, porosity and permeability rises with increase in the irregularity of the shape of individual particles.

The distribution of particles also influences the permeability of the ground. Thus, for instance, ground consisting of horizontally arranged laminated particles has lower permeability than ground consisting of identically shaped particles but placed vertically although porosity of ground may be identical in both these cases.

Cultivated grounds have a specific structure considerably increasing their porosity and permeability. Individual grains of a plowed soil join one another and channels of greater diameter are formed between them. That is why the quantity of precipitation permeating tilled soil is much higher than with uncultivated ground.

While discussing surface water losses, attention should be drawn to the fact that a decrease in surface runoff also occurs as a result of the ability of the terrain surface to retain water in minor ground depressions.

The retentive ability (or retention) of a catchment basin surface should be determined every time in the field; approximate values may be accepted from Table 14.

Table 14

Retentive Capacity Z of a Catchment Area

Surface	Retentive capacity Z in mm
Asphalted surface	2
Cobbled surface	6
Meadow with sparse grass, cultivated soil	10
Meadow with abundant grass, woods, sparse moss	20
Thick layer of moss	40

A curve of water filtration correlated with time is shown in Figure 23. The slope of this curve is different for various types of ground and, for impermeable surface (asphalt), the curve passes into a horizontal straight line.

Diagrams of runoff from impermeable and permeable surfaces are shown in Figure 24. Rain fallen on a surface with depressions approximately parallel to the contour lines may cause no runoff even on a slightly permeable ground.

It follows that pores, crevices and other empty spaces in the ground are

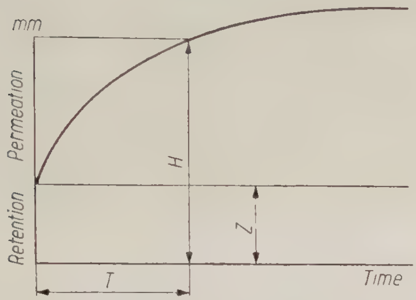


Fig. 23. Curve of correlation between infiltration and time

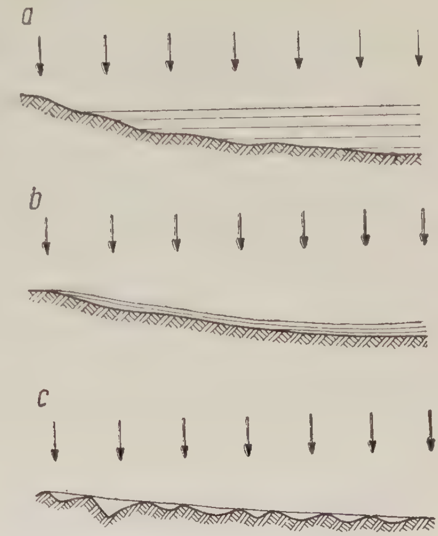


Fig. 24. Diagram of runoff: *a* — impermeable soil, *b* — permeable soil, *c* — terrain with oblong depressions

a kind of water reservoir, retaining water which gets inside the soil and ensuring a more uniform consumption of it. The greatest stores of ground waters appear in catchment basins consisting of permeable and absorbent grounds.

Appearance of Lakes and Marshes in Catchment Basins.

Lakes unconnected with a river network, or situated along the route of a river or constituting river sources, may appear in a catchment basin (Figure 25).

The first type of such lakes causes a decrease in runoff, because a certain part of the water gets into them from the catchment basin in the form of superficial or underground runoff and evaporates.

The second type are natural reservoirs, accumulating water during periods of increased flow and, subsequently, feeding rivers for a long time during low water stages. Thus, lakes contribute to the control of discharge all the year round which facilitates, for instance, the more rational exploitation of the water power.

The feeding ability of a lake depends on its dimensions, situation in a catchment basin, conditions of water inflow and flow downstream etc.

Under Poland's climatic conditions, lakes cause a decrease in the magnitude

of mean annual runoff consequent upon losses of certain quantities of water evaporating from the lake surface. Extreme discharges are still more affected by the influence of lakes. The larger the area of lakes in a catchment basin the lower the magnitudes of maximum discharges and the higher the magnitudes of minimum discharges.

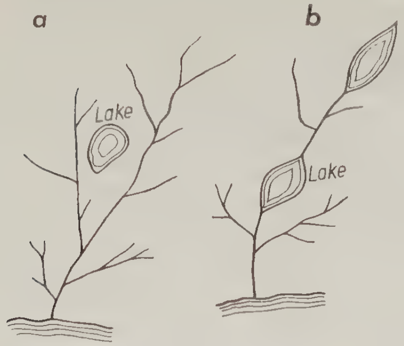


Fig. 25. Diagram of lakes located in a catchment basin:
a — lake not connected with a river network,
b — lake connected with a river network

Valley flats of a river, and particularly river valley extensions shaped like lakes, exert exactly the same influence on the water discharge as lakes do.

A controlling influence of a valley and a valley flat consists in the fact that some quantity of water reaches these areas during highwater stages, thus decreasing the discharge in a river. After a rise passes water runs off from the valley and valley flats into the river thus increasing the volume of discharge.

Water, reaching a valley flat not only fills that area, but also some quantity of it filters through the soil to ground water level. After subsidence, a part of this water gets back to the river by a subterranean route, thus increasing the magnitude of discharge at low stages and slightly extending the period of a rise.

The influence of marshes on discharge should also be examined. From the hydrological point of view, a marsh is a certain area of ground surface where soil is saturated constantly or for longer periods with water to a degree exceeding the moisture limit.

The influence of marshes on the magnitude of discharge is complex in nature and difficult to characterize.

It has been observed that the decrease in the annual layer of flow downstream takes place as a result of great quantities of water evaporating in marshy areas. At the same time, the magnitude of runoff per annum is regulated by the appearance of marshes. It has also been established that marshes contribute to an increase in summer runoff in average years as a result of water accumulation and feeding rivers by subterranean routes during the low water stages. In dry years, on the other hand, marshes dry up and, therefore, do not take part in feeding the catchment basin by way of ground waters.

Marshy terrain contributes to a more uniform discharge throughout the entire year. This characteristic of marshes has nothing in common with peaty grounds, such as are to be found in these areas, but it results from mild slopes of the marsh surface and from the appearance of clumps of trees and bushes, patches of ground with vegetal cover, together with other obstacles hindering a free runoff of water.

Vegetal cover is extremely important in forming runoff in catchment basins, influencing the magnitude of runoff and the uniformity of feeding water to catchment basins.

Not only the size of areas covered with woods, meadows, fields, etc., but also the situation of these areas in a catchment basin should be established accurately to characterize the vegetal cover of such basin. Also to be noted is whether individual kinds of vegetation cover a continuous area or are scattered and form separate patches.

The influence of the vegetal cover can be shown in a different manner, in some cases as decreasing the magnitude of discharge, in others as causing an increase.

The surface of soil covered with grass is marked by a greater roughness than of bare ground. It contributes to the delay of runoff and increase in the quantity of water filtering through the ground, which in its turn diminishes the surface runoff. Morever, grass absorbs from the soil water necessary for its vegetation, which subsequently evaporates. For these reasons, runoff is lower and more uniform in catchment basins covered with grass.

The influence of forest and bushes on discharge is more complicated.

Trees consume water for transpiration thus decreasing the ground moisture, i. e. depriving rivers of a certain amount of water which otherwise would be fed to them by the subterranean route. Forests and bushes intercept by their branches a part of the precipitation, which evaporates without reaching the surface of the ground.

The specific structure of soil in forests contributes to an inscrease in the intensity of the water filtration into the ground. Leaves, sod, green moss and various types of biological litter covering the surface of the forest soil delay the freezing of ground which, consequently contributes to more rapid water infiltration. Analogous effects are caused by roots of trees loosening the ground to considerable depths.

Together with such phenomena causing a decrease in runoff, forests and bushes, arising out of their many other properties, also contribute to an increase in the magnitude of runoff.

A well-known phenomenon, for instance, is that greater quantities of rains fall on areas covered with forests than on open fields. Forest litter covering ground surfaces plays the role of an insulating layer diminishing the intensity of evaporation of water contained in the ground.

Water infiltration may be considerably diminished by woods if an impermeable layer is formed within the ground under them as a result of the penetration of salts. In some cases, such a layer may even lead to swamping of the entire forest terrain.

It results from this, that the action of forests contributes to the change in quantity of the precipitation, conditions of evaporation, and conditions of runoff. Changes in the configuration of terrain are also caused by forests since a certain insignificant ground elevation may be observed around each tree.

It is difficult to decide how discharge will be affected by all the factors indicated. Therefore, considerable discrepancies in scientists' opinions can be noted, especially since many fundamental problems concerning the influence of forests have not yet been solved and explained.

There is a prevalent opinion that in principle, forests cause a decrease in maximum discharges, because they undoubtedly control the water regimen of rivers. They cause a decrease in spring rises, delaying the thawing of snow and thus prolonging the period of the rise. A similar effect is produced by forests in mountain and hilly areas.

5. River Overgrowth

On account of the development of vegetation in the channels of streams, various discharges, depending on the season of the year, can be passed at an identical elevation of water level.

The majority of investigators hold the view that the influence of vegetation overgrowing the river on the magnitude of discharge should be taken into account by the application of the additional coefficient K:

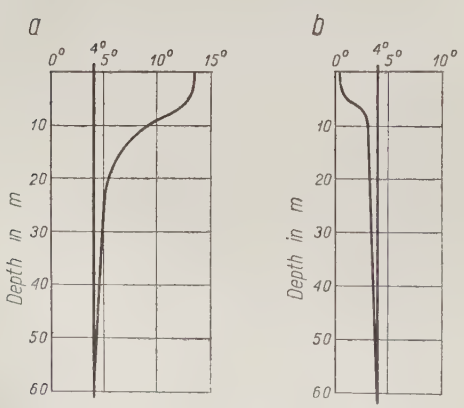


Fig. 26. Correlation between water temperature and depth:
a — in summer, *b* — in winter

$$K = \frac{Q_v}{Q_f}$$

where:

Q_v — discharge in an overgrown channel,

Q_f — discharge in a channel free of vegetation and ice.

It has been proved by investigations that a close relation exists between the development of the vegetal cover and changes in the temperature of water (Fig. 26). Since the correlation between temperatures of air and water is well-known, investigators usually avail themselves of the observations of air temperature, being easier.

For computing discharges, a collection of curves — separate for spring, summer and fall — was used by the Polish hydrologist, Faust. Such a computa-

tion should be considered as approximate because, if the number of measurements is sufficient separate curves for individual months are distinctly apparent.

On the basis of investigations, Faust reached the following conclusions:

(a) the influence of vegetation on the conditions of discharge disappears at temperatures between +6°C and ±10°C;

(b) along with a rise in temperature above this limit, the ratio of decrease in the discharge tends asymptotically to a certain value, different for each cross section and depending on the characteristic features of each such cross section;

(c) after a long period of low temperatures at the end of the summer, a sudden rise in temperature (called in Poland "the golden fall") does not affect the conditions of discharge, because it cannot revive the plants already nipped by sharp colds.

The conclusions of Faust comply approximately with those presented by Soviet hydrologists, who have come to the opinion that the beginning of the vegetation period of plants coincides with the period of mean daily temperature within limits of +4°C and +10°C, and the end — at temperatures ranging between +3.5° and +8°C.

To establish the coefficient K in practice, Ogievskii's method is convenient, as starting also from the formula presented above:

$$K = \frac{Q_v}{Q_f}$$

To compute the coefficient K, the discharge curve for the vegetation period is necessary, and also the curve for the period when the channel is not overgrown — i. e., between the end of the ice drift and the beginning of vegetation, and after the plants die to the appearance of the ice cover on the river.

Usually, the number of measurements during the latter period is insufficient for determining the correlation:

$$Q_f = f(H)$$

In that event, Ogievskii recommends the adoption of the curve limiting at the bottom of the graph, all the measurements of discharge plotted on it as a curve of discharge in a channel free of vegetation and ice. Then, all the points located above this curve will represent the discharges corresponding with a river channel overgrown with vegetation or covered with ice. To compute by means of this curve, the values of the coefficients K the curve of these coefficients:

$$K = f(T)$$

should be drawn, where T — number of days elapsed from the beginning of vegetation.

The final values of this curve are equal to unity. The curve of the coefficients K is usually drawn from measurements taken in a given cross section (Fig. 27). This, however can cause great deviations from the real values of the coefficients for individual years.

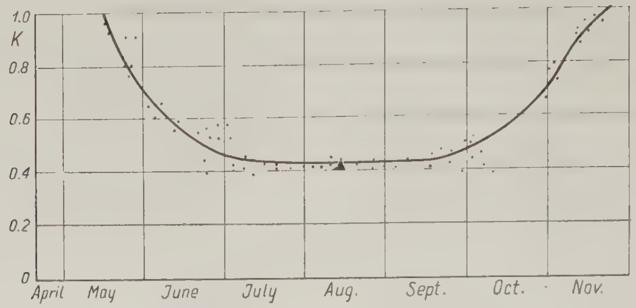


Fig. 27. Coefficient curve K for discharges in an overgrown channel

6. The Activity of Man

The quantity of flowing water and the regimen of a river can be changed as a result of various operations undertaken by men in a catchment basin, or between the catchment basins of adjoining rivers. Since these changes are sometimes of an essential character, a tendency has recently appeared to take them into account in hydrological computations.

It should be borne in mind, however, that so far there are no accurate methods making it possible properly to take into account the activity of man in every particular case.

All operations performed by man and contributing to changes in discharge can be divided into two groups. The first of these includes building hydrotechnical structures in a river or between rivers, the second — various works performed in a catchment basin.

Dams, dikes, sluices, embankments, etc. are among hydrotechnical structures changing the natural regimen of rivers.

Some of these structures are built for elevating water or forming artificial resorvoirs which accumulate water during the rises (Fig. 28). During the low water stages, this water can be used for irrigating soil, feeding water works, increasing depths of navigable rivers, etc.

Structures of this type not only change the regimen of runoff, but also the conditions under which feeding water to rivers occurs. The elevation of water level in a river or reservoir constitutes an obstacle in the flow of ground waters, and may even cause the permeation of water from the river or reservoir in the ground — i.e., the opposite phenomenon to that appearing in nature.

Reservoirs increase the surface of the water area and, consequently, greater quantities of water evaporate.

A particularly great influence is exerted on the river regimen by hydrotechnical structures, by means of which certain quantities of water can be passed

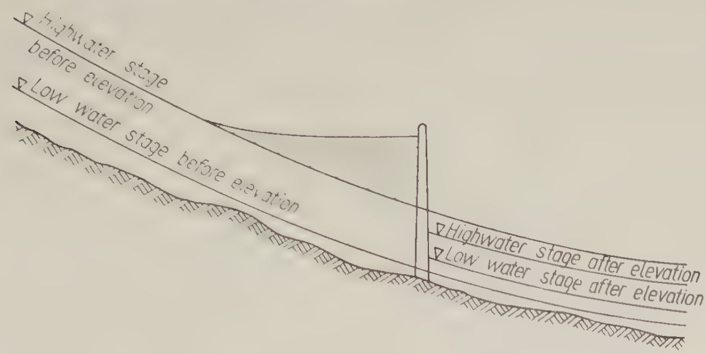


Fig. 28. Water surface before and after elevation

from one river to another. Such devices are installed, for instance, on the Moskva-Volga Canal.

Tree felling, afforestation, reclamation, etc. are among the second group of works undertaken in a catchment basin, which may cause a change in the water regimen of a river.

In the USSR, a certain influence on the change of the river regimen is exerted by new methods of soil cultivation, introduced to improve fertility. Thus, for instance, the soil is plowed there with the furrow parallel to the direction of river channels, protective forest belts are planted, etc.

It has been found, that in vast plowed up areas, furrows parallel to the direction of water movement in a river — i.e., approximately perpendicular to the direction of water runoff in a catchment basin — hinder water in its movement to feed the river. Investigations have shown that the coefficient of runoff in a terrain with furrows parallel to the direction of water in a river fluctuates between 0.006 and 0.018. while in fields with furrows at right angles, the value of the coefficient of runoff for an identical height of precipitation (10 to 50 mm) is contained within the limits of 0.056 to 0.201.

A decrease in runoff is also caused by planting protective forest belts, which retain the snow and create conditions favorable for water infiltration. At the same time, the snow thawing period is extended, delaying the runoff and reducing the magnitude of evaportaion.

In addition to precipitation and evaporation, there are many additional factors influencing the annual and multiannual fluctuations of the runoff, height of the flow layer, etc. Defining the influence of additional factors is more difficult than computing precipitation and evaporation, which can be numerically characterized. Future investigations will, however undoubtedly establish ade-

quate methods to facilitate determination of the quantitative influence exerted by additional factors; and that will contribute to increased accuracy in hydrological computations.

CHAPTER III

STUDIES AND MEASUREMENTS NECESSARY FOR COMPUTING THE SPAN OF A BRIDGE

1. The Scope of Studies and Measurements

Elaboration of the Hydrotechnical Data Necessary for Bridge Designing

The building or rebuilding of every bridge requires the elaboration of hydrotechnical data.

A complex of such data should in principle include the following:

(1) general situation plan of the river;

(2) detailed situation plan of the river;

(3) characteristic cross sections of the river;

(4) longitudinal section of the river;

(5) plan of the catchment basin area up to the site of the structure designed (for small catchment basins);

(6) hydrotechnical report together with computations.

Such data are necessary for establishing the maximum discharge in the cross section of a future bridge and for computing the size of the span.

Furthermore, the following matters should be explained and substantiated by this elabortaion:

(a) selection of the stream intersection,

(b) location of the bridge,

(c) height of the edge of the road embankment on the approach to the bridge,

(d) level adopted for the head room under a bridge (vertical clearance),

(e) selection of places for the navigable pass and the dimensions of passes,

(f) dimensions and constructions of icebreakers,

(g) type and height of the regulating structure and their distribution in plan.

The following should be performed in the field in preparing the elaboration:

(a) situation survey;

(b) measurements of cross and longitudinal sections of the river;

(c) measurements of water velocities and discharges;

(d) measurements of changes in the outlines of the channel and the banks;

(e) observations of the course of fluctuations of water level with special reference to the highest stages and those characteristic for ice drifts;

(f) observations of ice phenomena.

The regimen of a river cannot be thoroughly examined on the spot by a measurement party during a short stay in the field. Use should therefore be made of the results of observations and measurements taken in the field for hydrotechnical elaboration of an intended bridge, together with all available results of observations and measurements taken over a number of years directly in the cross section of the intended bridge or in the adjoining sites.

Introduction to the River Regimen

Prior to departure for the field observation site, the regimen of the rive, in question should be studied on the basis of existing observation material together with hydrological and meteorological publications.

It should first be established whether the summer or spring rises cause maximum discharges in a given cross section, because they will be valid in computing the bridge span.

The discharge in a river is the most important factor delineating its character. Water levels, velocity, power, etc. vary with changes in the discharge.

During periods when the river is fed only slightly with water — e.g. under the ice cover, or during summer months marked by low precipitation — water levels fall, while in the event of intensive water flow during spring rises or periods of high precipitation in summer, water stages rise to a considerable extent.

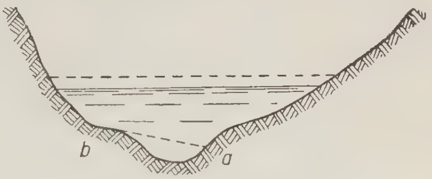


Fig. 29. Change in the height of the water level as depending on change in the river bottom:
a — bottom before deepening, *b* — bottom after deepening

Fluctuations of water level observed on rivers depend primarily on changes in discharge; a rise in water stages is usually caused by an increase in the discharge, while falls follow a decrease.

There are, however, many cases in which, during particular periods or in autumn, river sectors, fluctuations in water stages are accompanied by unchanged discharge. Such a situation may arise, for instance, following the lowering or elevation of the river bottom, river channel becoming overgrown by vegetation, the building of hydrotechnical structures, etc. (Fig. 29).

It is necessary to ascertain, therefore, on the basis of existing material,

that none of such changes in discharge conditions have taken place in a given cross section; adequate corrections should be introduced to ensure continuity of observations.

The winter regimen of a river should also be studied to determine the thickness of ice cover, dates when the river freezes, ice drifts, etc.

2. Measurements of a River Sector

Determination of the Peak Water Marks

Determination of the level and date of peak water is the most important operation during studies and measurements, since it will contribute to obtaining a more accurate value of discharge with an adopted probability.

Studies of this factor should be conducted both at the site of the projected bridge and, on occasion, dozens of kilometers along the river.

The peak water level in a cross section should be determined by several methods and the results plotted, for comparison and adjustment purposes, on a diagram of the cross and longitudinal section.

The peak water level may be determined from:

(a) archival materials,

(b) results of observation from water gaging stations,

(c) evidence of local residents,

(d) direct traces found on the spot.

Reports on former floods, losses and casualties inflicted by them and sometimes even mentions of rises previously occurring in such areas can often be found in archives, articles, and even in old newspapers. In working up such material, it is necessary to look for hints as to which areas were flooded, what was the level reached by the water, etc. It is then necessary to determine, by leveling, a water stage at which those floods appeared.

If results of observation of the water gaging station located near the site of an intended bridge are available, they should be used, but attention should be paid also to the zero height of a water gage, which might be subject to changes during the existence of the observation station.

Sometines this is a very difficult task, but it must be executed in spite of all difficulties because the acceptance of a water stage read from some other ordinate of the zero of the water stage may lead to entirely erroneous results.

If the results of observations over a longer period are not available, it is often necessary to establish the peak water marks on the spot. Indeed this method should be applied additionally even when multiannual observations are available, with a view to proving their reliability.

To establish on the spot the level and date of appearance of peak water,

data collected among local residents are often used, and particularly those obtained from older people, who lived for a long time in the vicinity of the intended bridge site and remember the course of one or several floods which at one time or other have affected that area.

Care should be taken in selecting the places at which older local residents are asked to point out the level of past highwaters. It is easier, for instance, to show the range of water on a gentle than a steep bank. If an adjoining village was flooded, it is easy to remember and point out a particular street that was flooded, or a door, window, etc. of a house which was reached by water. In such cases, areas should be selected where water flowed smoothly or stood still; otherwise witnesses may instead of pointing to the true peak water level indicate an exceptional one caused simply by waves.

Thus, water levels should be determined at several places and, if possible, on the both banks of the river, as well as above and below the site of a future bridge. The levels thus obtained should be plotted on the situation plan and marked on the profile surveys. If the individual values thus obtained are considerably divergent, it is advisable to check them once more in the presence of all witnesses, so that their information can be mutually confronted and agreed upon.

The peak water level may also be established by traces left on the river banks, grass, and branches of trees inside the valley flat, by sand deposits in rock crevices or split trees, by sediment in valley flats of mountain rivers, etc.

In addition to peak water stage, the date of its appearance should also be established by evidence gathered from local residents, because it is necessary to determine the frequency of such water stage and to compare it with other stages on the same or adjoining rivers. It is usually possible to fix the year and month, which is quite sufficient for the purpose.

Determination of Discharge Conditions under Bridges Nearby

Some data concerning bridges in the vicinity can prove helpful when available hydrological materials dealing with the cross section under study are insufficient.

The following materials and data concerning such bridges should be prepared:

(a) scheme of the bridge and the size of its span;

(b) distance from the bridge to the river mouth, and the catchment area;

(c) dates of appearance of and maximum and minimum water stages appropriate to the series of years used in computations, together with peak and lowest water levels during the period of ice drifts;

(d) water velocities;

(e) river channel cross sections under the bridge, and geological section of the terrain;

(f) discharges measured and introduced in computations;

(g) data concerning the type of bridge pillars, depth of foundation, construction and dimensions of icebreakers, realignment structures, etc.;

(h) general characteristics of the work of a bridge opening indicating whether such pass is sufficient, too small or too great for the discharge of water, navigation by vessels, rafts, etc.;

(i) general characteristics of the work done by icebreakers, realignment structures, etc.;

(j) dates and types of damage done to the bridge by floods.

Similar information should also be obtained concerning the work of dikes, dams, canals and other structures if there are any such in the vicinity of the section of the bridge.

Topographic Survey of a River Sector

A river sector survey is necessary to plot the plan of its channel and valley flat with indications of height, cross sections and longitudinal slope of bottom and water level at various stages, and above all at the highest known stage.

The elaboration of bridge hydrotechnical data includes drawing a general and a detailed situation plan.

The general plan includes the area within the limits of all possible river crossing points, while the detailed plan, with contour lines, is made on a larger scale only for the area on which the individual structures are to be designed and built.

Plan of the General Situation

This plan is prepared with a view to proving the correct choice of site for the future location of a bridge, and the choice of the cross sections of the river. The approximate situation of a river sector under study also should show the location of all possible river intersections examined and cross sections chosen.

A closed polygon with sides traced as near as possible to the limits of an area surveyed should constitute a basis for the general situation plan (Fig. 30).

The vertices of the polygon should be marked by wooden poles. The angles of the polygon are measured with a 30-second theodolite in two positions of the telescope. The ground elevation of the vertices of the polygon should be established according to the official leveling network, and several stable height marks placed nearby.

The accuracy of the polygon survey should comply with the following requirements:

(a) the longitudinal error in closing the polygon should be not greater than in 1 : 1,000;

(b) the angle measurement error in closing the polygon should be not

greater than $3s\sqrt{n}$ to $1.5s\sqrt{n}$ minutes, wherein n = number of polygon vertices and s = accuracy of the instrument expressed in seconds;

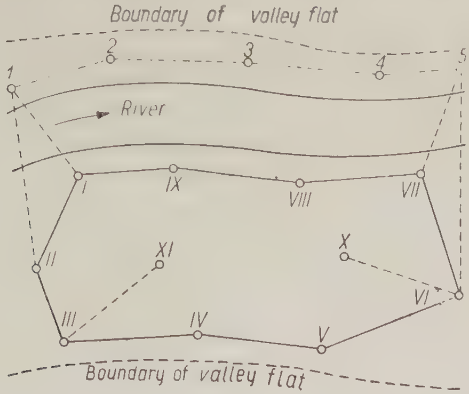


Fig. 30. Polygon for surveying the general situation plan

(c) the error in measuring the elevation should be not greater than 1 mm per 100 m of the length of the polygon in a flat area, or between $5\sqrt{n}$ and $10\sqrt{n}$ mm in a hilly terrain, whereas n = the number of instrument stations.

The situation survey is made by the tacheometrical method, and only general outlines of objects are surveyed — i.e.: boundaries of forests, bushes, marshes, water range in lakes, valley flats, etc. Elevation is measured only at characteristic points such as hills, deeper ground depressions in valley flats, along the coast line of a river channel, or bifurcation, etc. An example of the general situation plan of a river is shown in Fig. 31.

A general situation plan should be prepared for a river sector above the bridge at a distance equal to 1.5 times the width of the valley flat at the highest known rise, as well as below the bridge at a distance equal to the width of one valley flat. These distances should be doubled in mountatin and foothill areas.

The width of the sector surveyed should be equal to the width of a valley flat at the highest known rise, with a certain additional margin. In the mountain areas, this margin should be established by adding an area which would be flooded if the water level was 1.5 m higher than the level of the highest known rise.

The length of the river sector surveyed can be increased according to:

(a) the number of all possible alternatives of sites for a future bridge and their distances;

(b) the situation of the cross sections;

(c) the distribution of adjoining bridges, harbor and other structures influencing the selection of the route or the size of the bridge pass.

If there are several alternative sites which may be selected and the extreme

ones are at a distance not greater than three times the width of a valley flat, a survey of an entire river sector is prepared including all the alternative bridge sites.

If cross sections are selected outside the limits of the sector for which the situation plan should be prepared according to the assumptions indicated above, the scope of measurements should be correspondingly extended to include such cross sections in the plan.

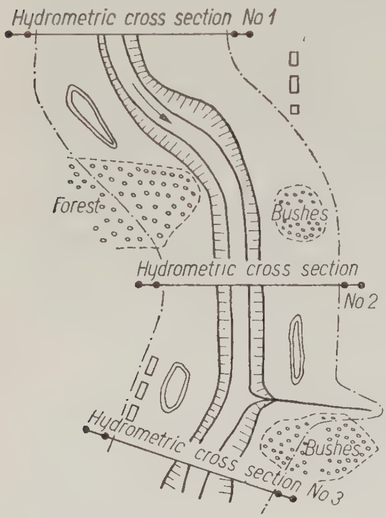


Fig. 31. General situation plan

If the river bifurcates into branches in the vicinity of the intended bridge thus influencing the discharge, a survey should be made of a river sector together with the branches.

The general situation plan is made in scales between 1 : 1,000 and 1 : 25,000, according to the size of the area surveyed. If maps of scale 1 : 25,000 are available for a given river sector, they can be used and all the details obtained on the basis of field measurements plotted on them.

The following details should be marked on the general situation plan:

(a) situation of all structures located in the area surveyed and approximate terrain relief, with elevation of characteristic places indicated;

(b) situation of river cross sections;

(c) limits of an inundated area at the highest known rise, as also limits of an inundated area caused by the backwater of another river if such a factor appears under given conditions;

(d) all the alternatives for a river crossing;

(e) course of the river, including all oxbows, branches, tributaries, mill-canals, islands, etc.;

(f) situation of buildings, roads, paths, forests, bushes, fields, meadows, etc.;

(g) water gages, height marks, triangulation tower, etc.;

(h) river course kilometer marks;

(i) regulation route with markings for regulative structures built and designed;

(j) survey network and coordinate values.

Detailed Situation Plan

This kind of plan serves to show an accurate survey of a river sector comprising the site of the intended bridge. The terrain configuration and distribu-

tion of all the existing structures should be shown as accurately as possible in the detailed plan, together with all the details of the situation of the river.

A detailed plan is necessary to draw the project of a river crossing, as well as to compute the scope of work; the surface surveyed should therefore be sufficiently large to hold the entire complex of structures envisaged.

A survey for the detailed plan should be made over the length of the river sector equal to about three times the anticipated length of the intended bridge above the bridge cross section, and 1.5 times the length of the bridge below such cross section. The width of the survey should be within the same dimensions as in a general plan.

If there is a possibility of shifting the axis of the route in the course of a detailed elaboration of a river crossing, the length of the river sector surveyed should be appropriately increased. The same principle is valid if there is a possibility of designing the realignment structures outside the limits of the area surveyed.

If the premises for the choice of an alternative in the selection of a bridge site are not entirely convincing, and other alternatives also have many advantages, measurements should be taken and detailed situation plans prepared separately for each alternative site if such are at a considerable distance one from another. If the distance between the corresponding cross sections is not large, all the alternative sites may be included in a simple plan.

The survey of terrain is made tacheometrically by means of a one-minute tacheometer; points should be so distributed as to achieve the most accurate picture possible of an area surveyed.

The route of the intended river passage, which should comply with the official existing level constitutes a basis for the detailed situation plan. The additional level circuit applied should be only the closed traverses — i.e. those both ends of which fit into the route of an intended river passage.

The accuracy of the assumption of additional circuits should be identical with the survey of the general situation plan.

To conduct the levelling in the area chosen for the survey of the situation plan, the height marks placed are connected with the official network by means of reciprocal levelling.

The detailed situation plan is made in a 1 : 500 or 1 : 1,000 scale for small rivers, and 1 : 2,000 for larger rivers. The 1 : 5,000 scale is applied to an area surveyed in excess of 10 sq km, provided that some details are worked up in a higher scale.

In the detailed situation plan, the same details should be shown as in the general plan, but with greater accuracy of presentation. The contour lines should also be drawn in the detailed situation plan with one-meter intervals between them. If the contour lines drawn at such distances do not present a distinct picture of the terrain configuration, additional contour lines should be drawn every 0.5 m or closer.

The volume of discharge is the quantity of water flowing through the transverse section of a stream per unit of time. To compute discharge, a measurement of a river cross section should be taken within the limits of a valley flat, such is called a hydrometric cross section.

The computation of the discharge should be made for checking purposes in several places at some distance one from another and, therefore, several hydrometric cross sections are chosen in the sector surveyed.

One of these cross sections is traced along the axis of the intended bridge. If there are in this place an old bridge or road embankments in valley flats, the cross section cannot be valid for computing the volume of discharge. In this event, an adequate cross section free of any structures should be selected in the vicinity of the bridge site.

The remaing hydrometric cross sections (the second and third) are chosen above or below the axis of the intended bridge in places favorable for surveying.

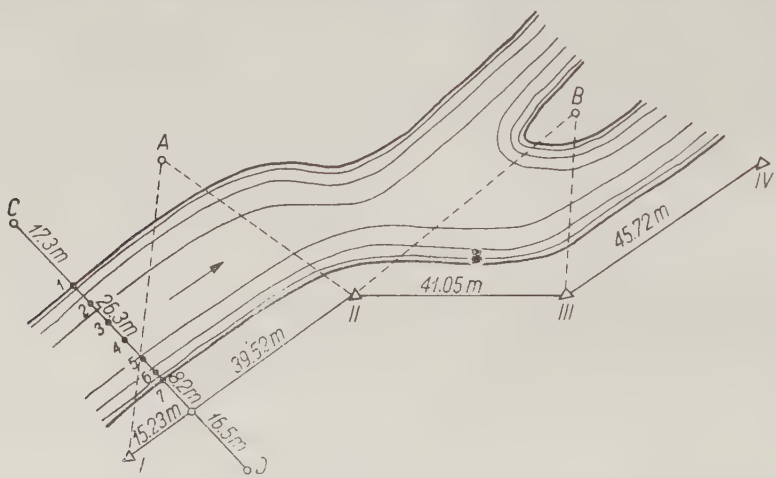


Fig. 32. Hydrometric cross section in connection with a polygon: *I, II, III, IV* — polygon, *A, B* — height marks, *C, D* — hydrometric cross section

If possible the river sector within the limits of which the hydrometric cross sections will be chosen, should be:

(a) rectilinear over a length equal to at least one width of the river for large streams, and to five times the width for small and average streams;

(b) without a valley flat — if unavoidable as is mostly the case in practice; then a compact river sector should be selected with high banks, without trees and bushes in the valley flat, without hills, etc.;

(c) without side branches, larger tributaries, islands, flooded rocks and other obstacles causing water elevation.

The river channel along this sector should in shape be approximate to a trapezoid or parabola, be equal in width and depth, and have a uniform longitudinal slope of the water surface along the entire sector.

After choosing the appropriate place, a hydrometric cross section is traced by means of a theodolite along a straight line, the length of which is measured twice with a steel measuring tape.

All the hydrometric cross sections should be connected by a polygon, which serves as a basis for the survey of the general situation plan. The situation of the hydrometric cross sections is shown in the plans, distances, angles, etc. being indicated (Fig. 30).

The terrain elevations in the hydrometric cross section are determined by leveling; the elevation of the local height marks should be very accurately referred to the official leveling network.

Thus, the terrain elevation may be established in a hydrometric cross section on both sides of a river channel, within the limits of inundation by the highest known water. The elevation of the bottom of the channel cannot be established. A measurement of the elevation of the water surface and the depth of the river at particular points of a cross section should be taken in lieu.

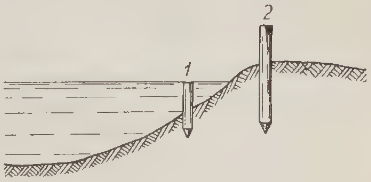


Fig. 33. Distribution of poles during measurements:
1 — pole necessary for the leveling rod, 2 — witness peg

The elevation of a water level, at which the depth of the river is measured, is determined as follows. Small poles are driven into the river bottom, their tops reaching the water surface in a cross section near the bank (Fig. 33). The leveling of these poles with the nearest height mark — at the beginning and end of the depth measurement — will indicate the elevation of the water surface. Appropriate corrections should be introduced if the top of the pole is not precisely flush with the water surface. A larger pole called a "witness" is driven into the bank near the measurement pole.

The depth of the river can be measured by several methods depending on the width and depth of the river, equipment available, etc.

Steel Tape Measurement by Fording

On small, shallow rivers, a steel tape is stretched along the axis of the selected cross section at a height of 0.5 m above the water surface, and the depth of the river is measured every 0.5 m or 1 m by an assistant fording the river and using a graduated measuring stick. A leveling staff is not recommended for this purpose.

This method is used for measuring depths of rivers wider than 200 m. A steel rope 4 to 8 mm in diameter with wooden or metal knobs fixed at intervals of 1, 2, 5 or 10 m is stretched at a height of 1 to 1.5 m above the surface of the water. White is usually used for knobs marking 1 m intervals, red for 5 m intervals, and blue for 10 m intervals.

The measuring party consists of a technician recording the results of measurements and 2 assistants, one of whom pushes the boat along the rope and the other measures the depths at intervals marked on the rope.

Fig. 34. Measurement of river depth from a boat by means of a rope

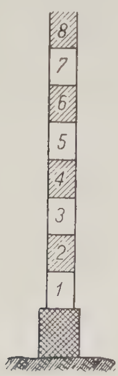


Fig. 35. Depth measuring sound

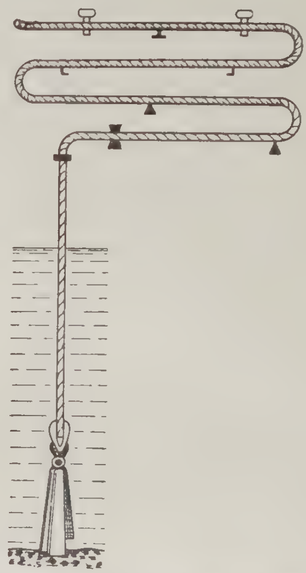


Fig. 36. Graduated rope for depth measurements

Usually, measurements are taken from the left to the right bank. The first reading along the rope is recorded in a place where the surface of the water touches the bank; the depth in this place amounts to zero. The boat is subsequently moved by an assistant along the stretched rope and stopped at every mark on the rope; then, the depth measurements are taken by the other assistant. The distances and the depths are recorded by the technician in the field diary. The last reading on the rope is recorded at a place on the opposite bank where once more the water depth is equal to zero.

The depth is measured with a wooden rod called a sound, on which decimeters are marked by notches and figures (Fig. 35). A five-meter sound is used

for water velocities between 0.5 and 1 m/sec; at lower velocities, a longer sound may be used if necessary.

In measuring depth, the sound should be thrown somewhat forward against the current so as to be in a vertical position after taking the impact of the water at the moment of reading the measurement.

Depths can also be measured with a hemp rope (Fig. 36) 3 to 5 mm in diameter with a weight of 4 to 6 kg (with high water velocities, up to 10 kg and heavier) suspended at one end. Decimeter intervals are marked on the rope with pieces of white cloth with red cloth tied to it. The upper surface of the weight suspended at the end of the rope constitutes the zero point of the graduation of the rope. Meters are marked by cutting slits in the pieces of red cloth for each full meter.

Since the rope is drifted by the current, a correction should be introduced, the magnitude of which is established by comparing rope measurements with sound measurements.

On large rivers, the use of a winch with a steel line 2-3 mm in diameter wound on it, and a streamlined weight attached to one end is recommended instead of manual measurement by hemp rope.

Note, that the depth measurement in bridge and hydrometric cross sections should be taken twice; differences in measurements should not exceed 2 percent. The distance between points at which depth is measured in bridge and hydrometric cross sections should not exceed:

0.5 m — on rivers up to 5 m wide,
1.0 m — ,, ,, ,, ,, 10 m wide,
2.0 m — ,, ,, ,, ,, 100 m wide,
5.0 m — ,, ,, ,, ,, 200 m wide,
10.0 m — ,, wider rivers.

Measurement from an Ice Surface

Taking depth measurements is easier and results more accurate when the river is icebound. The distances between points at which depth measurements are intended, are measured by a steel or linen measuring tape and marked on the surface of the ice. Then the ice is cut through at these places and measurements are taken — i. e., readings on the sound sunk in ice holes from the river bottom to the water level. To find the range of water at the bank, a channel is cut in the ice and the place where water touches the river bank is marked.

Topographic Measurement

On rivers wider than 200 m and with current velocity exceeding 1.5 m/sec, the location of the depth measurement points is determined by the tacheometer.

For this purpose, a base is established, which can be constituted by one side of a polygon or a sector of a straight line connected with the polygon; the length of the sector of a straight line (*AB* in Fig. 37) is measured with a steel tape.

A tacheometer is placed at point *A* and the initial measurement of the width of the river is made. For this purpose, leveling staffs are placed on both sides of a river, the distances of such from the instrument being established by means of a tacheometer. Then, depths are measured at certain intervals of time by an assistant in a boat moving from one bank to the other along the axis of a traced cross section; at the same time, angles are read by a technician operating the tacheometer.

The measurements may be organized as follows. Depths are recorded and measurement points on the river are established by a technician in a boat. When measurements are taken, signals are waved by an assistant with a colored flag. A technician operating the tacheometer and observing the movement of the boat through a telescope, records the reading and announces by a flag signal that the measurement is closed at a given point.

Usalluy, the location of all the points of the depth measurement are not determined by the tacheometer, but, for instance, only every fifth one, while the indirect location is established by means of a stop watch or by distances between particular strokes of oars.

Sometimes, two tacheometers placed at points *A* and *B* are used to increase the accuracy of measurements by simultaneous readings at each measurement point.

River Depth Determination by Measuring Longitudinal Sections

On large and deep rivers with considerable current velocities, river depth can be established by measuring longitudinal sections (Fig. 38).

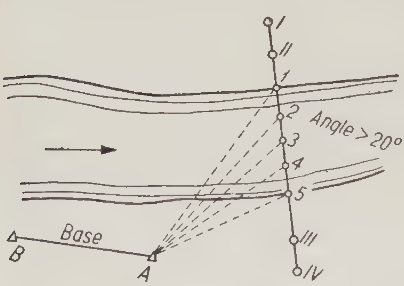


Fig. 37. Tacheometric measurement of river depth

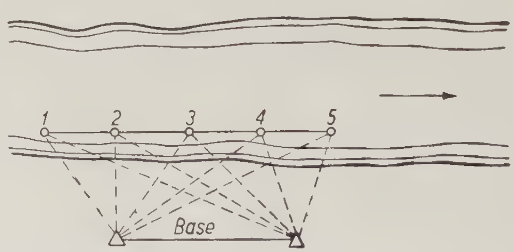


Fig. 38. Measurements of river depth by means of longitudinal cross section

First, the boat from which measurements are taken is allowed to drift freely in the current near the bank. A graduated measuring rope hangs from the boat

submerged in water to a depth about 1 m from the river bottom. At prescribed intervals of time measured with a stop watch, the rope is lowered to the river bottom and the depth of the river is measured. The individual measurement points are determined, as previously, by two tacheometers placed on the bank.

After arriving at the lower limit of a measured sector, the boat returns upstream; the downstream drift is then repeated, and measurements are taken as previously but at a different distance from the bank. Such measurements from the boat should be taken over the entire width of the river.

To speed up the work, it is advisable to use two boats: measurements are taken from one of them drifting downstream, while the other is returning upstream to the place where the next series of measurements are to be started.

Diagram of a Cross Section

A sketched example of a cross section of a river is presented in Fig. 39. The following elements should be marked on the diagram of a cross section:

(a) successive kilometers of a river course;

(b) elevation of individual points of terrain in correlation with the comparative level and results of sounding;

(c) highest water stage known by observation or established on the basis of evidence by reliable witnesses;

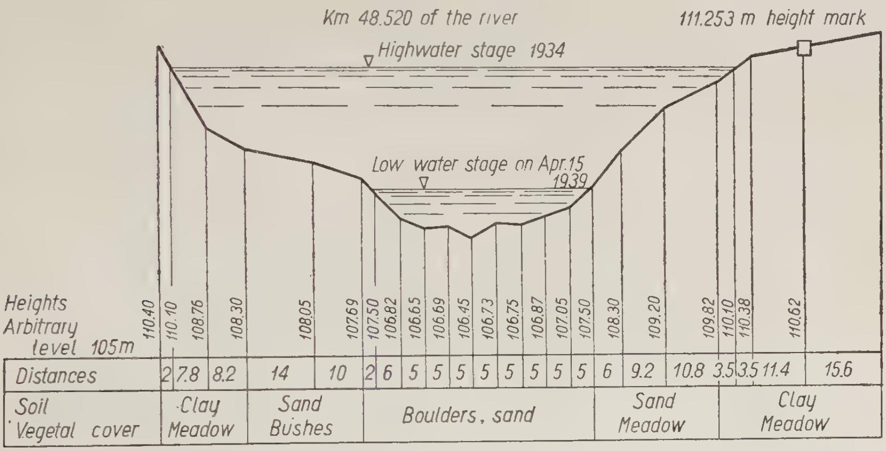


Fig. 39. Cross section of a river

(d) water stage during measurements;

(e) realignment structures, abutments, bridge piers, dikes and other structures influencing the discharge of water;

(f) staff gage or height mark, from which the cross- section survey was started;

(g) grade line of the carriage-way and vertical clearance of the bridge construction — if a bridge cross section is surveyed.

Data concerning the geological cross section of the terrain, and vegetal cover should also be given.

The scheme of various constructions of bridges with indication of places whose ordinates should be established by leveling are presented in Fig. 40.

Cross sections are mostly drawn in a distorted scale; the scale of depth (vertical) is usually assumed as 10 times greater than the scale of distance (horizontal).

Measurements Taken for Drawing the Contour Plan of the Bottom

A larger number of cross sections should be surveyed if depth measurements are taken for preparing the contour plan of the river bottom, or for determining the extent of the river channel scour. The distance between these cross sections must not exceed:

15 m — for river width up to 50 m,
25 m — „ „ „ „ „ 100 m,
50 m — „ „ „ „ „ 200 m,
100 m — for greater widths.

Cross sections are, if possible, traced-perpendicularly to the direction of

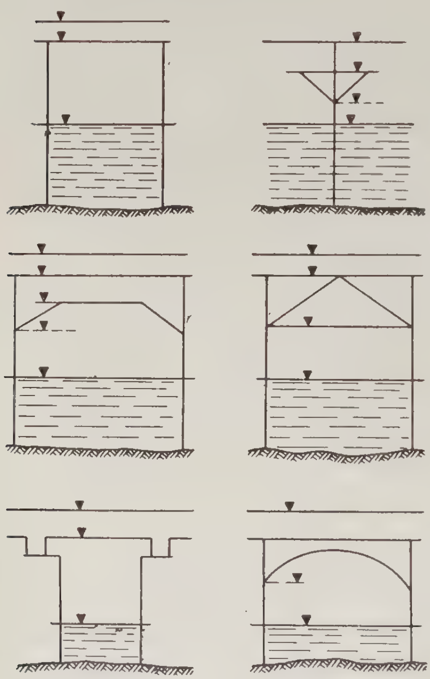


Fig. 40. Diagrams of bridges with leveled points indicated

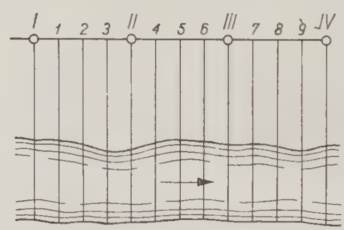


Fig. 41. Tracing cross sections

the channel line. The location of each cross section should be related to a polygon established on the bank. For this purpose, it is sufficient to measure the angle contained between the side of a polygon and the direction of a cross section, and the distance between the vertex of this angle and the nearest point of the polygon.

The sector of the cross section between the side of a polygon and the line tracing the range of water in a river channel should also be leveled. The distances of points at which heights or water depths are measured are counted from the side of the polygon.

At the points where cross sections bisect the sides of the polygon, poles should be placed, and marked with numbers according to the direction of the river current (Fig. 41). Further, a pole (two poles for larger river widths) with a bunch of twigs should be driven into the ground on each bank in the end sectors of a cross section. The bunches of twigs (or straw) should be large enough to be clearly visible from the opposite bank.

The distances between points of a cross section surveyed for drawing the contour plan of the bottom cannot be greater than:

2 m — for rivers up to 50 m wide,
5 m — „ „ „ „ 100 m „
10 m — „ „ „ „ 200 m „
20 m — for wider rivers.

For measurements taken to establish the degree of channel scour, these distances should be halved.

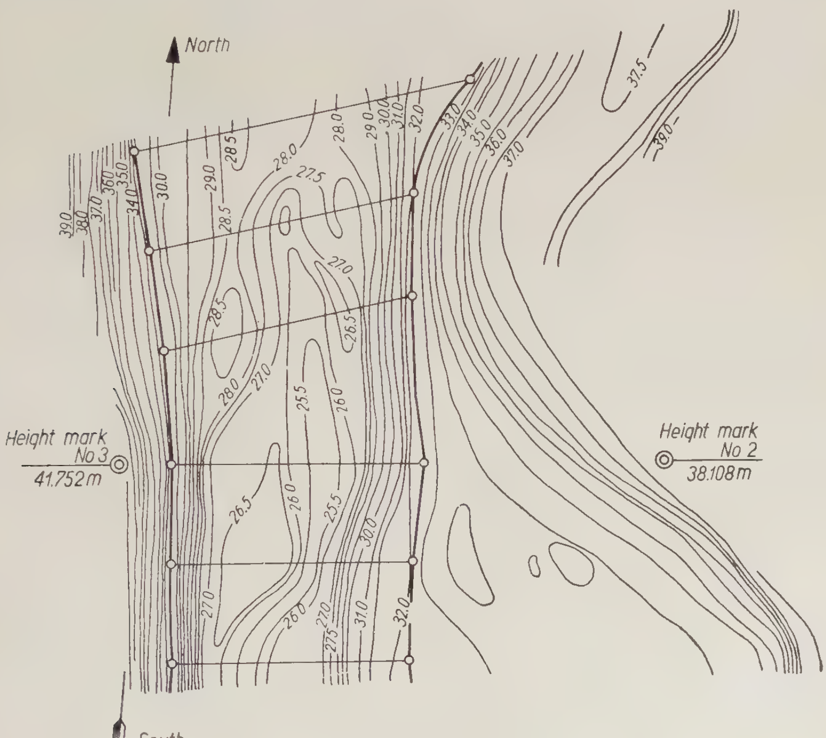


Fig. 42. Contour lines of a river bottom

The river depth should be measured at least twice, i. e. in winter, from the surface of the ice and after the spring or summer rise. The contour lines obtained from these two measurements should be plotted on the plan; to distinguish them, the former are marked by a continuous and the latter by a dotted line. A contour plan of a river obtained from the summer measurement is shown in Fig. 42.

Changes in the Shape of the Water Surface

Note, that sometimes, when cross sections are measured a convex water surface is obtained, while in other cases, it appears to be concave, a phenomenon usually ascribed to the inaccuracy of measurements. This ascription is, however, incorrect, because the shape of the water surface is actually subject to changes during rises and falls of the water level.

In the event of a sudden rise in the water level, an increase in the water velocity is observed in the central part of a cross section. In this connection, the pressure falls in this place and the water surface in the middle of the river assumes a convex shape. When the water level falls, an opposite phenomenon occurs and, in this case, the water surface in the central part of a cross section is lower; near the banks it is higher. On the basis of this phenomenon, it is possible to determine whether the water in a river is rising or falling. If small objects (rubbish, chips of wood, etc) freely drifting in water approach towards the banks, this is a sign of a water rise. If, on the other hand, they drift midstream, a fall of water is taking place.

Measuring the Longitudinal Slope of the Water Surface, Bottom and Banks

The longitudinal slope of the water surface in a river is the ratio of the difference between heights of the water surface (termed also gradients) at the beginning and at the end of a certain sector of a river to the length of such sector. Slope is usually denoted by a decimal fraction; for instance slope $i = 0.001$ means that the gradient amounts to 1.0 m per 1 km.

Measurements of longitudinal slope of the water level in a river are necessary for:

(a) computing maximum discharges with low frequencies;

(b) transposing the characteristic water stages from neighboring cross sections to the site of an intended bridge;

(c) computing coefficients of roughness for the channel and valley flat of the discharges measured;

(d) establishing ordinates of the crests of the realignment and other structures, etc.

To measure the longitudinal slope of the water level, small poles with their

tops standing 2-3 cm out of the water are first driven into the ground along one of the river banks. The distances between poles are to depend on the local conditions; it is necessary to drive poles in places where sudden changes of the longitudinal slope of the water level occur and in other characteristic places. To minimize the influence of waves on large rivers, it is desirable to place the poles in small channels dug in the soil of the bank and connected with the water in the river.

After driving all the poles, the leveling and measurement of the ends of the poles protruding from the water is performed.

The method of simultaneous measurement of the height of all poles if often used with a view to increasing the accuracy of the water level measurements. Using this method, poles are driven below the water level and a leveling instrument is placed near each pole.

Then, at a signal e. g. a shot or at an exactly predetermined time, nails are simultaneously driven into the top parts of all the poles to such a distance as to even the heights of the nail heads with the water surface; then, the height of the nail heads equivalent to the height of the water surface is immediately, leveled. With this method, it is possible to achieve a degree of accuracy of measurements within limits of $\pm$ 1 mm.

Water level should be read from the water gage staff simultaneously with taking measurements.

Besides measuring the height of the water level, leveling should be applied to the highest water marks in a given sector as fixed on the basis of evidence by local residents or according to traces left by a rise.

Sounding for greatest depth, which — plotted on the diagram — represents the elevation of the bottom, should also be undertaken in all river cross sections passing through points serving for measuring the height of the water surface. Further, the heights of both banks should be leveled.

The measurements of cross sections can also be used for drawing the longitudinal section of a river, provided that such measurements are sufficient in number. If these measurements were taken over several days, water stages might fluctuate during that period. In such cases, the results of all measurements should be reduced to a single level of water by means of simultaneous leveling of the poles driven into the ground in these cross sections.

The best results of measuring the longitudinal slope of the water surface are obtained at low water stages at a time when there is no wind or waves. The same length of the longitudinal section as the length of a sector surveyed is accepted for the detailed situation plan.

As a river longitudinal section is usually drawn for the channel line of a river, and the height of the water is measured at one bank only, it is advisable to introduce a correction of the height of the water level at the river bends. It is a well-known phenomenon that the water level is higher at the concave than at the

convex bank. The differences in heights of the water level, in a cross section situated on a river bend can be measured in the field or computed by the following formula:

$$h = \frac{v^2}{2g} \log\left(i + \frac{b}{r}\right)$$

where;

v — mean water current velocity,
b — river width,
r — radius of a bend,
i — river slope.

Diagram of a River Longitudinal Section

The longitudinal section of a river is drawn on the basis of measurements, the scale of length being accepted within limits of 1 : 1,000 and 1 : 10,000, and scale of height — 1 : 10 and 1 : 100.

The longitudinal section of a river should contain:

(a) comparative level;

(b) kilometer sectors and distances of the route of the river;

(c) line of the channel bottom at its deepest place;

(d) lines of the right and left banks;

(e) water level during measurement and its slope with the date of measurement indicated;

(f) level of the highest known water, with its slope and year of appearance indicated;

(g) water gage site with the height of its zero point indicated;

(h) situation of hydrometric cross sections;

(i) heights of existing height marks;

(j) places of tributaries;

(k) existing and projected realignment structures, with their highest points indicated.

An example of a longitudinal section of a river is shown in Figure 43.

3. Determining the Discharge by Water Velocity Measuring Instruments

The velocity of water movement is measured mostly for computing the discharge in a river. The measurement of water movement may be necessary for other special purposes such as, for determining the direction of a single water stream in a river.

Water movement in natural streams is a very complex phenomenon. Velocities in a stream are subject to changes during water level fluctuations, and also depend on the shape of the water surface in a plane, differences in slopes, con-

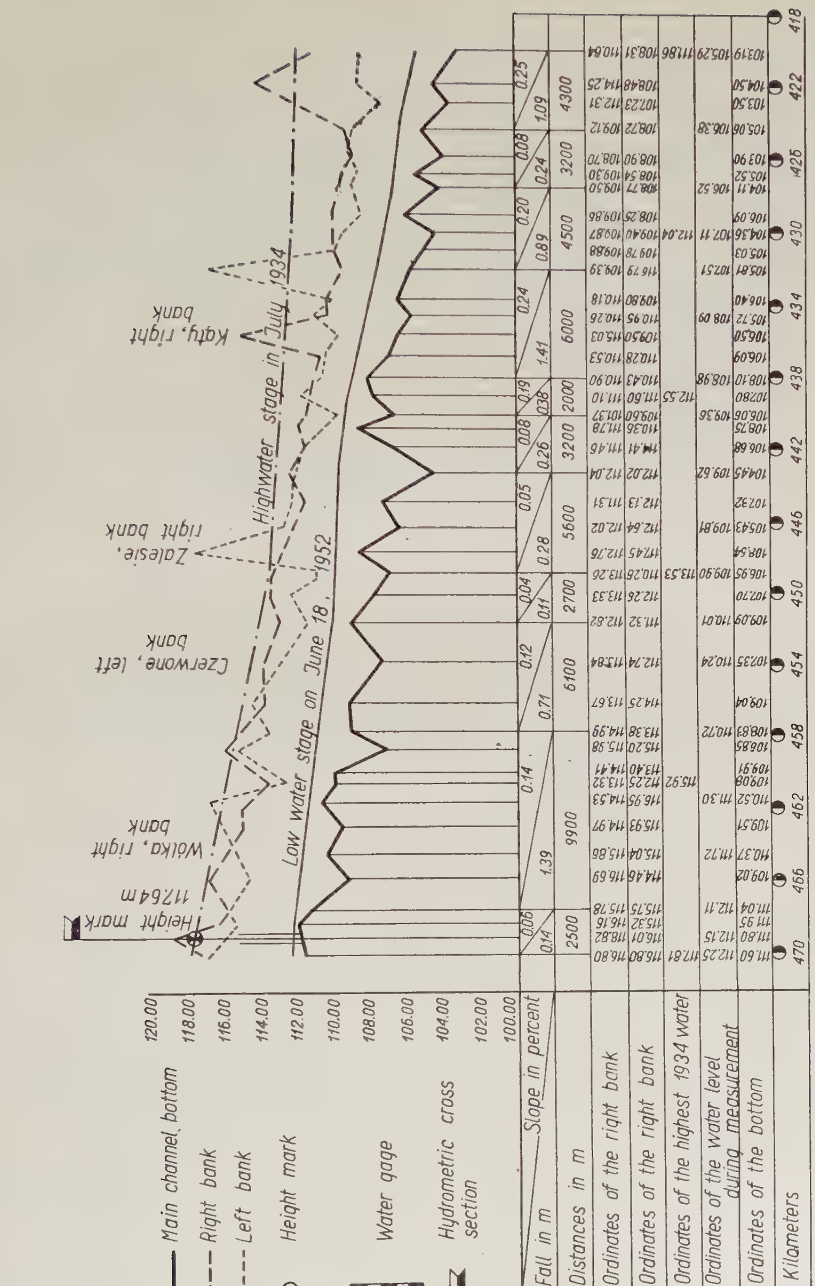


Fig. 43. River longitudinal section

figuration of bottom and banks, water depths, etc. Various velocities may therefore appear in various cross sections and points within the limits of a river channel and valley flat.

At every point of a cross section, at a given moment, there is distinguished in addition to the local velocity whose magnitude and direction changes at short intervals, an average velocity. This is a mean velocity of all individual, variable, and temporary velocities appearing over a certain period.

Figure 44 shows a diagram of temporary changes (pulsations) of velocities observed by Velikanov in the Channel Changes of the Research Laboratory of the USSR Academy of Sciences Geographical Institute. Temporary changes in velocities have been recorded by extremely sensitive electrical instruments called thermohydrometers.

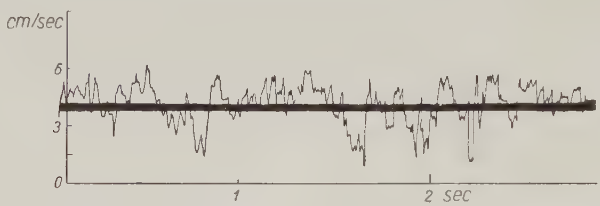


Fig. 44. Diagram of pulsations of moving water

The measurement of water velocity actually consists in directly establishing an average velocity at various points of a stream, because, in contrast to temporary velocities, it has a comparatively constant value. In the subsequent parts of this book, average velocity will be called simply — velocity.

Distribution of Velocity in a Cross Section

As already indicated, velocity varies considerably in individual sectors of a free flowing river, and at particular points of a cross section. Even so, there is a certain correlation between the velocity and the width and depth of a river.

Thus, for instance, velocities increase away from banks towards the middle part of the cross section of a river. The greatest velocities are to be observed in the deepest places; in valley flats they are much lower than in the main channel of a river. Velocities measured along a vertical line decrease towards the bottom.

The correlation between velocity and depth in a vertical can be shown graphically. For this purpose, points where water velocities have been measured are plotted on a vertical line the length of which is equal to the depth of a stream. Then, horizontal lines, equal in lengths to the velocities measured at such points, are traced through them. A curve thus obtained and connecting the ends of sectors is called a curve of velocity in a vertical, and the entire diagram is termed a graph of velocity.

Since instruments now in use do not make it possible to measure velocities at the very bottom of a river, the bottom sector of the curve of velocity is obtained by extending this curve down to the bottom. A similar method is applied for drawing the sector of the curve of velocity near the surface of water. True superficial velocity remains unknown, because instruments so far in use for measuring velocities require immersing in water.

A graph of velocities in a vertical of a free channel is shown in Fig. 45, and a graph of velocities under the ice — in Fig. 46. There are considerable differences between the graphs of water velocities in free channels and under ice because the velocities of water flowing under the ice cover decrease towards the bottom and lower surface of ice.

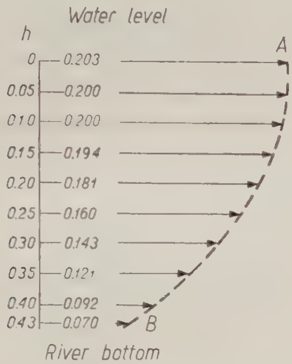


Fig. 45. Diagram of velocity in a free channel on a vertical

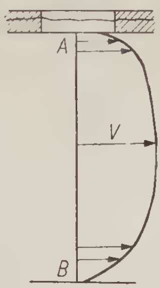


Fig. 46. Diagram of velocity under ice

The shape of graphs of water velocities in a vertical can be subject to changes depending on local conditions (Fig. 47). If the shape of a velocity graph departs

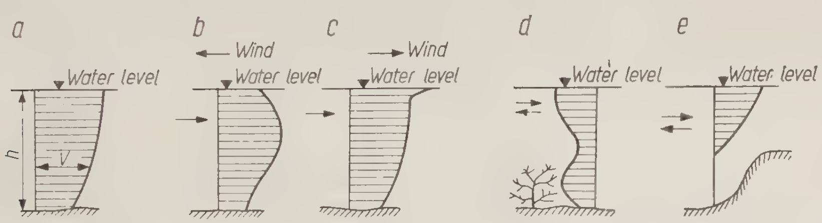


Fig. 47. Changes in the diagram of velocity in a vertical, as depending on local conditions: *a* — quiet water, *b* — wind contrary to the direction of the water current, *c* — wind directed along the current, *d* — valley flat overgrown with bushes, *e* — sudden elevation of the bottom

notably from the normal, the cause of the divergence should be found, because sometimes it may be the outcome of defects in measuring instruments.

The distribution of velocities in a cross section can be shown by the lines

of equal velocities — i. e., by isotachs drawn on the basis of velocities measured in the individual verticals of a cross section. Figure 48 shows isotachs in the period when a river is free of ice, and Figure 49 — isotachs for water flowing under ice cover.

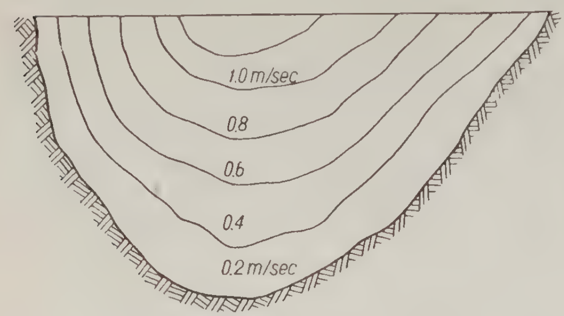


Fig. 48. Isotachs in a free channel

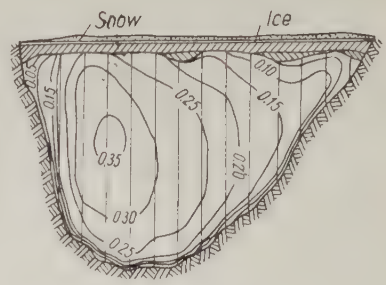


Fig. 49. Isotachs under the ice

Mean Velocities in a Vertical

In computing discharges on the basis of measured velocities, a mean water velocity should be determined for each vertical. It is possible to compute by way of mean velocity the discharge in particular parts of a cross section, when the sum of such discharges will represent the discharge over the entire river cross section.

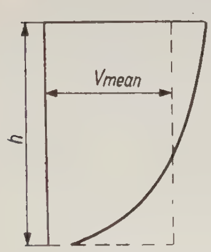


Fig. 50. Mean velocity in a vertical

The mean velocity in a vertical is an arithmetic mean of velocities at all points along the vertical (Fig. 50).

Investigation has established that, to draw a fairly accurate graph of velocity in a vertical, it is sufficient to measure velocities at a few points of such vertical.

Most appropriate is to select points for measurement of velocity in the following places:

(a) on the water surface;
(b) at a depth of 0.2 h from the water surface (h equaling depth of stream);
(c) at a depth of 0.6 h from the water surface;
(d) at a depth of 0.8 h from the surface;
(e) at the bottom.

If there is no need for particular accuracy, the measurements may be confined to the velocity measured at three points of a vertical, and even to a two- or single-point measurement.

The following methods are generally used for computing the mean velocity in a vertical:

(a) graphic mechanical method;
(b) graphic analytical method;
(c) analytical method.

The graphic mechanical method consists in determining the surface (P sq m/sec) of the velocity graph in a vertical by means of a planimeter, when dividing the surface area (sq m/sec) by the depth of the stream (h m) will give the mean velocity:

$$v_o = \frac{P}{h} \text{ m/sec}$$

The graphic analytical method consists in dividing the graph of velocities into several horizontal fields of identical heights, and reading the magnitudes of velocity in the center of each field. An arithmetic mean of such velocities will give the value of a mean velocity in a vertical:

$$v_o = \frac{v_1 + v_2 + v_3 + \ldots v_n}{n}$$

The analytical method of computing the mean velocity in a vertical does not require the drawing of a graph of velocities. The magnitude of a mean velocity is computed by one of the following formulas, depending on the number of velocities measured in a vertical:

(1) the following formula is used for measuring velocities at five points:

$$v_o = \frac{v_p + 3v_{0.2\text{h}} + 3v_{0.6\text{h}} + 2v_{0.8\text{h}} + v_d}{10}$$

(2) for measuring velocities at three points:

$$v_o = \frac{v_{0.2\text{h}} + 2v_{0.6\text{h}} + v_{0.8\text{h}}}{4}$$

(3) for measuring velocities at two points:

$$v_o = \frac{v_{0.2\text{h}} + v_{0.8\text{h}}}{2}$$

(4) for measuring velocity at a single point:

$$v_o = v_{0.6\text{h}}$$

The most accurate results are obtained by computing the mean velocity with five points; less accurate is the use of two points (2-3 percent error); and finally — three points.

Least accurate of all, in fact, is the computation by measuring at a single point, but it is of considerable importance in practice. It has been stated on the basis of numerous observations that the velocity measured at a depth of 0.6 h

from the surface of water is approximately equal to the mean velocity in a vertical; here, the divergence from the actual velocity amounts to about 6 percent.

Considerable differences may arise in an analytical method of computation if the velocity graph is of irregular shape.

One of the first three formulas, depending on the number of measurement points, can be used for computing the mean velocity of water flowing under ice. If a measurement is made at a single point only, the mean velocity of water under ice is computed by means of the following formulas:

$$v_0 = 0.85\, v_{0.4h} = 0.85\, v_{0.5h} = 0.90\, v_{0.6h}$$

During rises, ice drifts, etc., the mean velocity in a vertical should be computed by measuring the superficial velocity (v_p), because, in these cases, difficulties arise when measuring individual velocities in a vertical. A coefficient should then be introduced, the magnitude of which may be best determined by direct field measurements. The following approximate value can be accepted:

$$v_0 = 0.85\, v_p$$

The mean velocity in a vertical v_o can also be computed from the Matakiewicz formula, using the superficial velocity v_p measured in a vertical:

$$\frac{v_0}{v_p} = 0.78 + 0.015\,\mathrm{h} + \frac{0.02}{i^{0.7}}$$

where:

h — numerical value of the depth in a vertical in m;
i — numerical value of the slope of water level per mil

Floats

Float and current meters are mostly used for measuring water velocity in a river.

There are surface, depth and integrating floats. Surface floats serve for measuring velocities on the surface of water; depth floats — for velocities at some depth from the surface; and integrating floats — for measuring mean velocity in a vertical.

Surface Floats

The simplest surface floats can be made of a wooden disk 10-20 cm in diameter and 3-6 cm thick, cut out of a dry tree trunk. Bottles partially filled with water and sealed are also used for this purpose. The quantity of water poured into the bottle should be such as to allow only the bottleneck to protrude above the surface of water. On large and deep rivers, floats of considerable size may be used, made of two boards or rods 50-100 cm long nailed together and weighted with stones (Fig. 51).

For better marking, floats are sometimes painted white or red and provided with colored flags.

Water Velocity Measurements

Water velocity measurements by means of floats should be conducted in still, windless weather and at stabilized water level. For this purpose, a straight river sector should be selected with approximately identical depths and widths. In length it should correspond to about four times the width of the river. There should be no vegetal cover near the banks, or other obstacles which may cause deviation of water streams.

Three hydrometric cross sections — upper, central (main), and lower — are established at identical intervals on the selected river sector. The distance

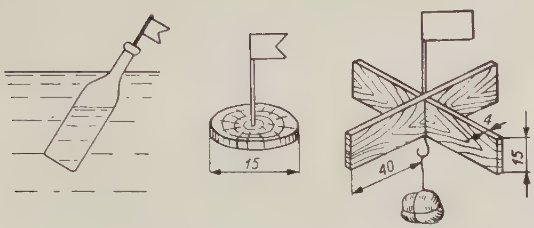


Fig. 51. Surface floats

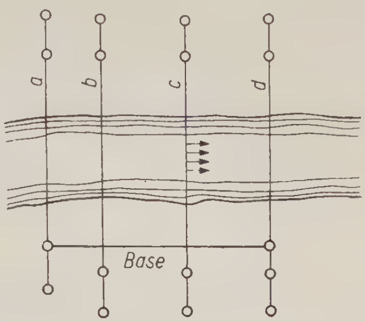


Fig. 52. Location of cross sections for measuring velocities by floats

between extreme cross sections amounts in meters to about 30 v_m (where v_m — maximum velocity in a river). An additional cross section is selected at a distance of from 5 to 10 m above the upper cross section. Floats are launched on the water in the additional cross section so that they may have gathered the velocity of the stream they are drifting with when they arrive in the upper cross section (Fig. 52).

Several observers should participate in measuring velocities by floats. On small rivers, particulary when ropes are stretched along the cross sections, only two observers are necessary, one of whom launches the floats in an additional cross section, while the other observes the movement of a float and, walking along the river bank, marks the place on a rope and on a stop watch checks the time taken by the float to pass through the individual cross sections.

On wide rivers, from 4 to 5 observers are necessary, one of whom launches the floats from the boat and another marks on a rope the places at which floats pass through the individual cross sections. If a tacheometer is used for determining these places, flag signals are waved by the observer when a float passes

a cross section. Other observers, using stop watches, check the time taken by the float to pass through the individual cross sections.

Floats are launched on the water, if possible, at identical intervals 5 to 8 points of a cross section. The observations of water stage fluctuations should be made on the nearest water gage staff. The mean velocity of the movement of a float is computed by means of the formula:

$$v = \frac{L}{t}$$

where:

v — mean velocity of current in m/sec conditionally related to the point of the central hydrometric cross section passed by the float;

L — distance between the upper and lower hydrometric cross sections in m;

t — time in sec required by the float to traverse the route between cross sections.

The accuracy of the measurement of velocity using a surface float amounts to from 8 to 15 percent.

Computing Discharge

The volume of discharge can be computed by means of the velocity measured using surface floats launched on the whole width of a river, applying the following method:

(1) the cross section of a river in the central (main) hydrometric cross section is plotted on tracing paper; depths and distances from the measurement base selected on the bank are written at the bottom of the graph (Fig. 53);

(2) the successive numbers of floats are written on the graph of a cross section in those places where they passed the line of the cross section;

(3) the floats which passed near one to another are connected on the graph into groups, and mean distances of each group from the measurement base are determined;

(4) floats whose velocities differ over 10 percent from the adjoining ones are disregarded in each group; the measurements of floats numbered 5, 9 and 19 have been discarded in the example given;

(5) the mean velocity as a quotient of distances of extreme cross sections and time are computed for each group of floats;

(6) the velocities thus computed are plotted on the graph, upwards from the surface of water;

(7) the points obtained are connected by a continuous line, representing the graph of surface velocities on the width of the river;

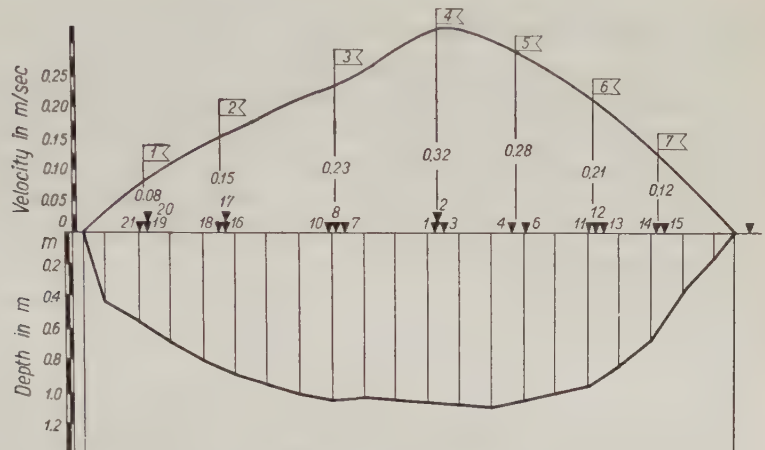
(8) the surface velocities for each vertical in which water depth has been

measured are determined by this graph and the values are entered under the graph;

(9) the areas p of individual fields, between the adjoining verticals, are computed from the formula:

$$p = \frac{h_{n-1} + h_n}{2} b$$

where b — distance between verticals, h_{n-1} and h_n — respective depths;



Depth in m	0	0.46	0.56	0.70	0.82	0.90	0.96	1.04	1.06	1.04	1.06	1.08	1.10	1.10	1.05	1.03	0.98	0.84	0.58	0.36	0.18	0
Distance from base in m: to verticals	10.6	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48		51.7
Distance from base in m: to floats			14.4			19.3			26.3			32.7			42.5		42.5		46.5			
Velocity in m/sec: to floats			0.08			0.15			0.23			0.32			0.28		0.21		0.12			
Velocity in m/sec: in verticals	0	0.04	0.07	0.10	0.13	0.16	0.18	0.20	0.23	0.25	0.29	0.32	0.32	0.30	0.28	0.24	0.21	0.18	0.13	0.08	0.03	0

Fig. 53. Diagram for measuring discharge by means of floats

(10) the surface velocities are computed for each field by means of the velocities in verticals:

$$v = \frac{v_{n-1} + v_n}{2}$$

(11) the areas p are multiplied by velocities v, giving the approximate discharge in each field;

(12) the approximate discharges are summed up in each field giving an approximate discharge over the entire cross section.

The discharge is termed approximate because it is computed by multiplying the areas of cross sections by the surface velocity and not by the mean velocity. To obtain the real value of the discharge, the approximate discharge should be multiplied by coefficient K:

$$Q = KQ_1$$

where;

Q — real discharge;

Q_1 — approximate discharge;

K — coefficient, whose mean value can be accepted as 0.85.

If the current meter measurements are taken in the main hydrometric cross section, they should be used for finding the accurate value of the coefficient K.

Measuring Maximum Surface Velocity

On small rivers with high water velocities, or if measurements have to be conducted quickly, such measurements may be confined to determining the maximum surface velocity only.

For this purpose, some 10 floats are launched in the water in the middle of the river and the two which float the distance between two cross sections in the shortest time are selected.

The highest surface velocity is an arithmetic mean of the velocities of the two selected floats, but the difference between their respective velocities must exceed 10 percent.

Using this method, there is no need to determine the distribution of points at which the floats pass the hydrometric cross sections; ropes, too, are, therefore, unnecessary.

The computation of mean velocity in the entire cross section by means of the highest surface velocity measured can be performed by the Matakiewicz formula:

$$\frac{v_s}{v_p} = 0.59 + 0.02\, t + \frac{0.006}{i}$$

where:

v_s — mean velocity in m/sec,

v_p — highest surface velocity in a cross section in m/sec,

i — numerical value of the local slope in promille,

t — numerical value of the mean depth in a cross section in m.

The magnitude of the mean value for the entire hydrometric cross section may also be determined by multiplying the maximum surface velocity by the coefficient K_o:

$$v_o = K_o\, v_{max}$$

The magnitude of the coefficient K_o can be computed from the following empirical formulas:

$$K_o = \frac{v_{max} + 2.354}{v_{max} + 3.129}$$

or

$$K_o = \frac{C}{C + 14}$$

where:

v_{max} — numerical value of the highest surface velocity in m/sec;
C — coefficient from the Chezy formula.

Depth Floats

A depth float consists of a surface float supporting and indicating the movement of an immersed float connected with it by means of a line or chain. The surface float can be made of boards, and the iinmersed float of barrels, wooden balls, etc. (Figs. 54a, b).

Since the upper (surface) float is much smaller than the lower (immersed) one, the pressure exerted by single water streams on the former and on the line can be disregarded, assuming that it indicates the velocity of river streams at the depth where the immersed float moves.

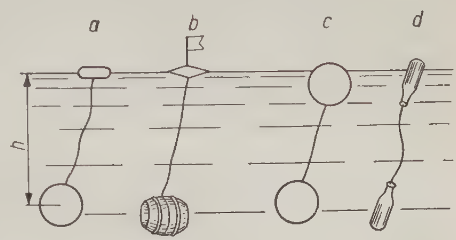


Fig. 54. Depth floats

The immersed float is usually lowered to a depth of about 0.6 h because mean velocity in a vertical is thus obtained at once.

Measurements of velocity at some depth are more accurate when double floats are used, consisting of two floats of approximately identical size and weight (Figs. 54c, d).

Using this method, the velocity on the surface should first be measured by means of a separate surface float and, subsequently, by a double float.

The velocity v_o of the movement of the double float can be assumed as being equal to the arthmetic mean averaged from the velocity v_p of the surface float movement and the velocity v_z of the immersed float movement:

$$v_o = \frac{v_p + v_z}{2}$$

A correlation

$$v_z = 2v_o - v_p$$

is obtained from this equation.

Since the lower part of a double float is usually immersed to the depth of 0.6 h from the water surface, therefore:

$$v_z = v_{0.6h} = v_o$$

After obtain ng the mean velocity in a vertical, a discharge can be computed

in a field adjoining such vertical. The sum of discharges thus determined will produce the magnitude of discharge over the entire cross section.

Integrating Floats

An integrating float (integrator) gives more accurate results than other types of floats. It consists of a ball, 2 to 4 cm in diameter and made of a material whose specific gravity is lower than 1 — e. g. wax, stearin, celluloid, wood, etc. This ball is attached by a thread to a string which is passed through a ring fastened at the lower part of a wooden rod (Fig. 55).

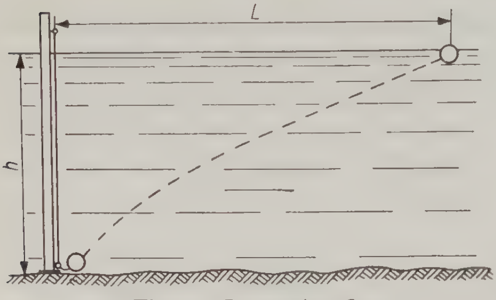


Fig. 55. Integrating float

When the wooden rod is immersed in water and the string pulled the thread breaks, releasing the ball which starts to float to the surface, covering at the same time a certain distance downstream.

As the ball rising from the bottom passes all the points distributed at various depths of the stream, it acquires a velocity equal to the mean velocity in a vertical.

The mean velocity in a vertical in m/sec is computed by means of the following formula:

$$v_o = \frac{L}{t}$$

where:

L — distance in m from the point of submersion of a float to the place where it emerges from the water;

t — time in sec from breaking the thread to the moment the float appears on the surface of the water.

An integrating float, after it has been rated, can be used to determine the mean velocity in a vertical by measuring only the distance L and without measuring time t.

Rating the float consists in lowering it to various depths in still water and computing the velocity u of its return to the surface:

$$u = \frac{h}{t_1}, \qquad t_1 = \frac{h}{u}$$

If the value t_1 is substituted for t in the formula $v_o = \frac{L}{t}$, another equation is obtained:

$$v_o = \frac{L}{t} = \frac{u}{h} L = CL$$

A table of values of the coefficient C for various depths (e. g. every 0.1m) is worked out as a result of rating an integrating float; such a table serves for the determination of the mean velocity in a vertical by means of the measured length L.

An integral float gives accuracy of measurement within limits of 5 and 8 percent at current velocities reaching 0.2 m/sec; at higher velocities, readings of distances and times are difficult.

The Use of Floats for Establishing the Direction of Current in a River

The Observations of the direction in which water currents flow are necessary for:

(a) correct location in a plan of a bridge axis and realignment structures;
(b) determining the influence of a stream on particular parts of a bridge cross section;
(c) determining the necessary size of navigable passes of a bridge and their location in a plan.

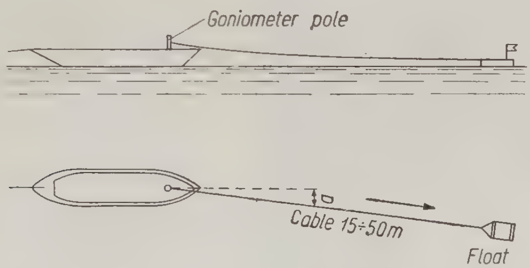


Fig. 56. Measuring stream direction by means of a float

The measurement of the direction of water streams can be effected by marking the situation of a float in several places by means of a tacheometer placed on the bank. Floats similar to those used for measuring surface velocity can be used for this purpose, they should, however, be larger to make them clearly visible.

On small rivers or in overgrown valley flats, the direction of streams can be observed by means of a float attached to an anchored boat (Fig. 56). Special goniometers (angle gages) are used for determining angles of deviation of currents.

Current Meters (Vane Counters)

A current meter (vane counter) is the most accurate and preferable instrument for measuring water velocity in rivers. It consists of three main parts: vanes, body and rudder.

Working Principles and Construction of Meters

The working principle of a current meter consists in the fact that pressure of water flowing at a certain angle to the surface of meter vanes immersed in water causes them to revolve. The number of revolutions of vanes depends on the velocity of the water.

The vanes of a meter consist of two or more blades which can be helically twisted or shaped like windmill vanes. Individual types of current meters can differ as to the length of vanes, which usually vary between 2 and 10 cm. Longer vanes turn in a more uniform manner, but they cannot be used for velocity measurement in shallow waters near the bottom or near the surface because the vanes must be fully immersed in water during measurements.

The body of the current meter is usually streamlined, causing less water turbulence during measurement. The shaft, usually mounted on ball bearings, and other devices are enclosed in the body.

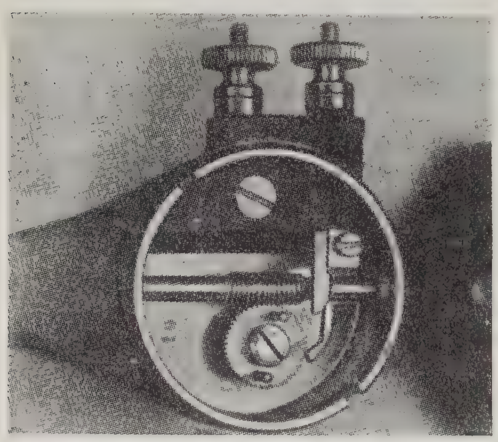


Fig. 57. Commutator of current meter

A commutator (Fig. 57) serves for counting the number of vane revolutions. A worm, mounted near the cut of the shaft drives a cog-wheel provided with one, two or more brads. When the cog-wheel turns, the brads successively take up a position in which they contact a spring connected with the pole of a dry battery.

An electric contact produces a flow current which sets in motion the signalling devices (buzzer, ball, etc.). The number of vane revolutions per second can be determined by recording the number of signals (e.g. of a buzzer) corresponding to the defined number of vane revolutions (e.g. every 25 revolutions), and the time, when measurement is made, according to the stop watch.

A correlation exists between the number of vane revolutions and water velocity and, therefore, knowing the number of the revolutions the velocity of water can be determined.

An automatic counter of signals, or a self-recording device, can be switched into the electric current circuit. In this case, there is no need for direct computation of the number of vane revolutions.

To take measurements, the current meter is attached to a special rod or suspended on a cable and subsequently immersed in water to a required depth. Some meters are equipped with devices facilitating either of these methods of fastening; small meters, however, are usually attached to a rod and large ones are suspended at the end of a cable.

A rudder commonly consists of a metal plate placed vertically fastened to the body of the meter and facilitating the adoption by the meter of a position parallel to the direction of the current.

The velocity at which the vane begins to turn, or the sensitivity of the meter, depends on its construction. It is advisable that the meter vane should begin its revolutions at water velocities not higher than 0.03 m/sec.

To make it possible to measure velocities by means of a current meter, a correlation should be established between the number of revolutions of the vane and the velocity of the current. This is achieved by rating current meters in a rating tank.

Every meter should be rated prior to being used because uniform friction of the mobile parts and identical conditions for their operation cannot be ensured during the production of the meter. The meter should also be rated after about 40 measurements, because bearings abrade during their work, with a possible resulting change in the correlation between the number of vane revolutions and the water current velocity.

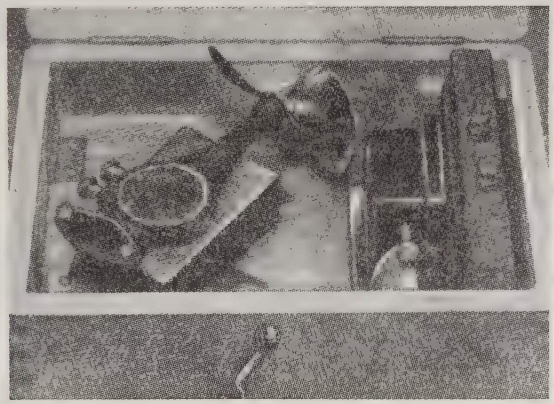


Fig. 58. Current meter, made in Poland

In Poland, the Polish made PE — EL type (Fig. 58) of current meters are now mostly used. The sensitivity of the small meters of this type — called pocket current meters — amounts to about 0.05 m/sec and, therefore, they cannot

measure lower velocities. Larger, 6-kilogram meters of the same type — sensitivity about 0.10 m/sec and 25-kilogram meters — sensitivity of about 0.20 m/sec — are also in use.

Figure 59 shows the Gurley cup type meter and Figure 60 — a winch which serves for suspending meters when measurements are taken from a bridge, gangplank, etc.

Fig. 59. Gurley type sensitive current meter

A current meter is a delicate instrument and should be carefully handled. The vanes and the shaft of a meter should be protected from damage since any possible bending of these parts can distort the results of measurements.

After taking the measurements, a current meter should immediately be taken to pieces and cleaned. Particular attention should be paid to cleaning the spring of the electric contact. If the balls of the bearings on which the shaft of the meter revolves are made of steel, they should be taken out after the work is completed and stored separately, greased with vaseline. When the meter is reassembled, the balls should, if they are going to work immediately, be rubbed dry, or if the meter is not to be used for some time, grease should be left on the surface of the balls.

In winter, a wet meter taken out of water can become covered with ice, which must not be removed by knocking, scrubbing, etc. but by plunging the whole instrument into hot water. Meters do not freeze if they are washed with salt water as soon as they are taken out of the river.

Water Velocity Measurements by Current Meters

Water velocity measurements are taken in several verticals selected in a discharge section line, the number depending on the width of the river (Table 15). The distances between individual verticals are usually smaller close to banks and to the water course.

Velocity measurements in every vertical are mostly taken at five characteristic points, the distances of which had been previously indicated.

Observations show that the velocity of a stream in any place is subject to constant changes (pulsations) of short duration within limits up to 30 percent.

Fig. 60. Winch for measuring velocities from a bridge

As a change of short duration develops from the surface of the water towards the bottom and from the center of the cross section towards the river banks, the time taken for the velocity measurement at various points of the cross section

Table 15

Number of Verticals Depending on the Width of the Channel

Channel width in m	1–5	5–20	20–50	50–100	100–300	300–500	500–1000
Number of verticals	3	3–5	5–10	10–15	15–20	20–30	30–40

should be varied. Usually, the time taken for measurement in a verical cannot be shorter than 5 minutes at the bottom, 4 minutes at a depth of 0.8 h, 3 minutes at a depth of 0.6h, and 2 minutes at a depth 0.2h, and on the surface.

The order of operations in measurements taken by a current meter is as follows:

(a) time of observation and water level read at the nearest water gage is recorded;

(b) water depth is measured twice in a vertical under study, and the mean depth is assumed as valid;

(c) depths at individual points of a vertical, selected for the velocity measurement (e.g. at 5 points) are computed;

(d) water velocity is measured by means of a current meter at first on the surface and then, successively, at the remaining points of a vertical.

Before measurements are begun, a record is made in a field diary of the date of making the measurements, the situation of the discharge section line and nearest water gage, the water level, conditions of the bottom, meteorological conditions (weather, wind), characteristics of the current meter (type, number of vane revolutions), etc.

Recorded in the course of measurements are the distances and depths in verticals, depths of the immersion of current meter, number of vane revolutions during measurements, and times checked on a stop watch.

Computing Discharge by Means of Measurements with a Current Meter

There are several methods of computing the discharge on the basis of velocity measurements taken with current meters. The most often used are: the analytical, graphic-analytical, mechanical-analytical, and Culman methods.

With the analytical methods, the areas of separate parts of a cross section limited by adjoining verticals are first established. The mean velocities in the individual verticals are then determined by means of the empirical formulas already presented, according to the number of points in a vertical.

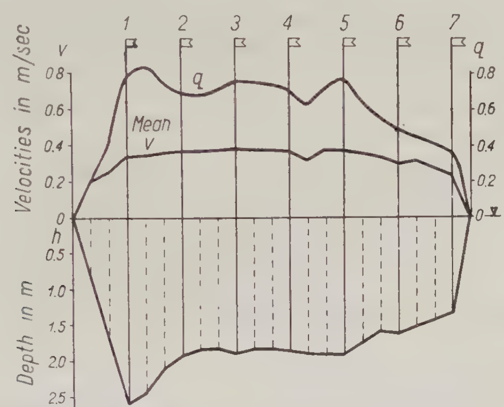
The discharges for each separate part of a cross section are obtained by multiplying the magnitude of areas between verticals by one half of the sum of mean velocities in such verticals. The sum of all the discharges thus obtained represents the volume of discharge over an entire cross section.

A general collection of data should show discharge Q, area of a cross section F, river width B, largest depth h_{max}, highest velocity v_{max}, mean velocity in a cross section $v_o = \frac{Q}{F}$, number of verticals and points in verticals, mean depths $h_o = \frac{F}{B}$, longitudinal slope of the water surface above and below the hydrometric cross section, wetted perimeter P, and hydraulic radius $R = \frac{F}{P}$.

It should be remembered that the wetted perimeter of the stream cross section under the ice cover is equal to the wetted perimeter in a free channel, together with the width of the water surface (because of the contact between water and the surface of ice). In computing the hydraulic radius for the cross section under ice cover the length of the cross section of ice is also taken into account.

Graphs of the discharge section line and velocity in verticals should be drawn for computations made by the graphic-analytical method. The following is the order of procedure when this method is used:

(1) a cross section of the river is drawn to a convenient scale on the basis of depth measurements, and the depths h are entered below it at points of individual measurements (Fig. 61).



Distances in m	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	26.0	28.0	30.0	32.0	34.0	36.0	38.0	40.0	42.0	44.0
Depths h in m	0.90	1.81	2.60	2.44	2.10	1.91	1.84	1.83	1.95	1.86	1.85	1.88	1.90	1.89	1.90	1.74	1.60	1.64	1.50	1.42	1.36	0.00
v m/sec	0.20	0.26	0.33	0.35	0.36	0.37	0.38	0.38	0.39	0.31	0.39	0.38	0.32	0.38	0.39	0.37	0.35	0.30	0.31	0.29	0.27	0.00
q sq m/sec	0.18	0.41	0.75	0.85	0.76	0.70	0.70	0.69	0.76	0.75	0.72	0.71	0.61	0.71	0.74	0.65	0.56	0.49	0.47	0.41	0.37	0.00

Fig. 61. Graphical-analytical velocity computation

(2) velocity graphs for individual verticals are prepared, and mean velocity for each of them is computed by means of the previously described graphic-analytical method;

(3) the mean velocities thus computed are plotted in particular verticals of the graph, to an adopted scale, from the water surface upwards; the points thus obtained are connected by a line representing the graph of mean velocities over the width of the river;

(4) the mean velocity v in all points of depth measurements is computed

from this graph, the values obtained being entered below the discharge section line;

(5) the values of the elementary discharge computed in points where the depth measurement is made is a product of depth h and velocity v, i.e. q = hv sq m/sec;

(6) the values q thus obtained are entered below the graph, and plotted in a selected scale above the water surface in the discharge section line; the resulting points will, when connected by a curve, produce the graph of elementary discharges;

(7) on the basis of the graph of elementary discharges, the discharges for individual fields between verticals at which depths are measured, are computed from the following formula:

$$\Delta Q_n = \frac{q_{n-1} + q_n}{2} b$$

where:

q_{n-1}, q_n — elementary discharge in adjoining verticals in sq m/sec;

b — distances between the verticals in m.

The discharges for the end sectors are computed from the formula:

$$\Delta Q_o = \frac{1}{2} q_1 b_1$$

(8) an integral discharge Q for the entire cross section is determined from the formula:

$$Q = \frac{1}{2} q_1 b_1 + \frac{q_1 + q_2}{2} b_2 + \ldots + \frac{q_{n-1} + q_n}{2} b_n + \frac{1}{2} q_n b_{n+1}$$

The formula for computing discharge can be simplified if the distances are identical between particular verticals at which depths are measured, i.e.: $b_1 = b_2 = b_3 \ldots . = b$.

To simplify the computations, a certain degree of inaccuracy in determining discharges for end sectors is usually tolerated in practice. In this case, the integral discharge is computed as a sum of elementary discharges $\sum q$, multiplied by the distance between verticals b:

$$Q = b \sum q$$

The sum of elementary discharge $\sum q$ is computed in this case by adding together all elementary discharges entered below the graph of the discharge section line.

For computing discharge by the graphic-mechanical method the same graphs are used as for the graphic-analytical method described above. The only difference consists in the fact that mean velocities are determined by applying

plane geometry to the vertical graphs of velocity. To obtain an integral discharge, plane geometry is applied to the area limited by the line of the graph of elementary discharges and the line of the water surface. Measuring appropriate areas by plane geometry should be performed with extreme care.

The graphic-mechanical method of determining discharge is time absorbing, but it is considered to be the most accurate, provided that plane geometry is carefully applied.

It is most advisable to compute the discharge by the less complicated graphic-analytical method; the graphic-mechanical method can be applied to check the results.

When applying the Culmann method, the points representing velocities read from the graphs are plotted on the corresponding depths in individual verticals of the river cross section.

The fluent curves passing through the points of identical velocities are called isotachs or curves of equal velocities (Fig. 62).

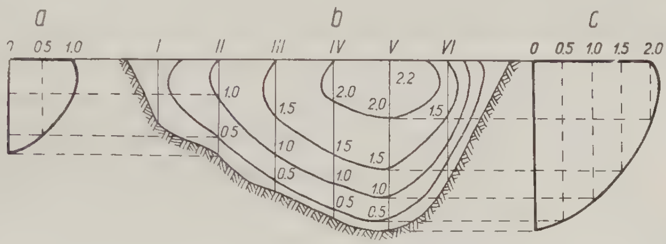


Fig. 62. Culmann method of computing discharge:
a — diagram of velocities in vertical *II*, *b* — river cross section with isotachs drawn, *c* — diagram of velocity in vertical V

Planimetry is then applied to the individual areas F_i between isotachs, as well as between the bottom or banks of a river channel and the nearest isotachs.

Mean water velocities v_i in these areas are equal to the arithmetic mean of velocities expressed by the adjoining isotachs.

The volume of water flowing per second through the corresponding areas of a cross section are obtained by multiplying F_i by v_i, the sum of these volumes representing the full discharge over an entire cross section of a river:

$$Q = \sum_{1}^{n} F_i v_i$$

where:

Q — full discharge in cu m/sec,
F_i — areas contained between isotachs in sq m,
v_i — velocity between isotachs in m/sec,
n — number of particular areas between isotachs.

4. Auxiliary Computations and Diagrams for Establishing the Span of Bridges

It often happens that some additional work not discussed in the foregoing chapters should be performed in establishing the maximum discharges. Such types of work include the determination of the river basin area and drawing the discharge curve.

Computing the Catchment Basin Area

Catchment basin area should be computed on maps in the scale of 1 : 100,000, and higher scales with the contour lines marked. The use of maps in a lower scale is inadmissible for this purpose, since it may give only approximate results. The catchment basin area of smaller rivers are computed on maps in the scale of 1 : 25,000, while the areas of catchment basins embracing a few or some dozens of square kilometers may best be measured directly in the field by the usual methods described in textbooks of geodesy.

To compute the catchment basin area, the watershed lines should be found on a map, and the area limited by this line should be measured.

The area of a catchment basin is measured on a map by means of a planimeter; small catchment basins should be traced several times around with a planimeter and a mean value adopted as valid.

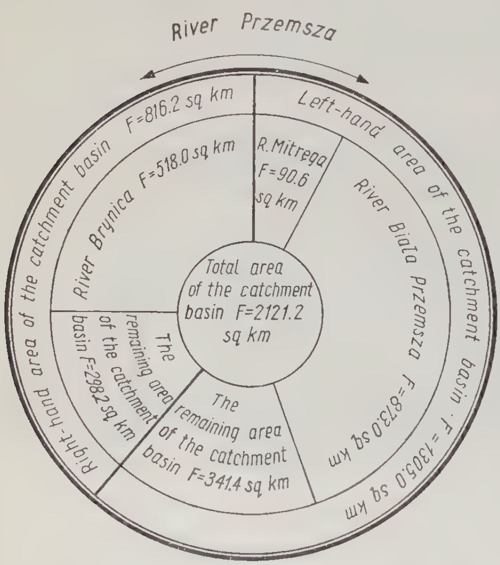


Fig. 63. Circular diagram of the Przemsza River catchment basin

Larger areas are planimetered in stages by twice tracing each part of the catchment basin with a planimetr. The sum of particular areas situated within the limits of one geodetic trapezium should be equal to the exact area of such trapezium presented in appropriate tables. The possible error is distributed in proportion to the area of particular fields as measured with a planimeter.

The results of the computation of the area of the main river basin and its tributaries can be shown graphically so as to give a clear view of the whole. A circular diagram of a catchment area, in which the total area of the catchment basin is shown in the form of a circle, and the area of particular tributaries in the form

of sectors drawn in a corresponding scale, is particularly clear and intelligible (Fig. 63).

Discharge Rating Curves

A water stage is read from the water gage during the measurement of water velocity for the purpose of determining the correlation between discharge Q and water stages H:

$$Q = f(H)$$

The line representing the correlation between these magnitudes is called the discharge curve or the rating discharge curve.

The discharge curve is drawn in the following way: the water stages above zero of the gage are plotted in an agreed scale on the axis of ordinates, and the discharges corresponding with them — on the axis of abscissae. The points thus obtained are connected by a line which usually approximates to the shape of a parabola.

The upper part of the rating curve usually has no points, since the discharge measurements are rarely taken at high water stages. For this reason, particular points of the upper branch of the rating curve are supplemented by computations. The respective values of discharges are mostly obtained by multiplying the area of the stream cross section by the velocity computed by empirical formulas.

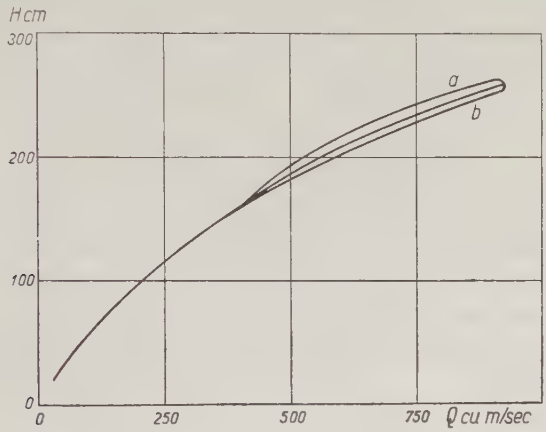


Fig. 64. Discharge loop curve:
a — for falling water stages, *b* — for rising water stages

The part of the discharge rating curve for the low water stages is usually drawn on the same diagram but to a larger scale, so as to obtain more accurate

readings, since the lower branch of the curve usually slopes more steeply than the upper part.

The discharge rating curve may be also expressed by the following equations:

$$Q = aH^2 + bH + C \qquad Q = a\,(H + b)^n$$

The graphical method should be applied, because it involves a sufficient degree of accuracy, the construction and use of the curve being easier than the deriving and use of an equation.

The magnitude of the mean velocity and of the discharge depends on the longitudinal slope of the water level. Continuous fluctuations of water stages in a river appear under normal conditions, the slopes at rising stages being larger than at falling stages.

Therefore, discharges measured at the same water stages can differ depending on whether the measurement was taken during the rise or the fall of water. This is the reason why two curves of the discharge are sometimes drawn — one for the rising and the other for the falling water stages.

A discharge loop curve is shown in Fig. 64. The upper part of it is valid for determining discharges at the levels of a rising water and the lower one — of a falling water.

CHAPTER IV

COMPUTING MAXIMUM DISCHARGES WITH AN UNKNOWN PROBABILITY OF APPEARANCE

1. Review of Methods in use

A discharge which appeared at the highest stage observed by the water gaging station or indicated by local residents has been considered thus far to be valid for computing bridge spans or culvert openings on a river. The period during which the maximum discharge thus determined appeared was not taken into account. Therefore, it was an accidental discharge with an unknown probability of appearance.

The most accurate method of determining maximum discharge with an unknown probability of appearance consists in direct measurement of water velocity at the highest stage and computing an area of the stream cross section at that stage.

Water stages, by means of which it is easy to establish the stage of occurrence of a maximum discharge, are measured every day in a cross section near the water gage.

On the other hand, the measurements of the discharge are seldom taken in the water gage cross section because every measurement of discharge takes considerable time and requires both the attendance of trained workers and the use of special instruments.

The measurement of discharge at high water stages is the most difficult to prepare and carry out and such measurements, therefore, exist only for a small number of cross sections.

If a maximum discharge has not been directly taken, it may be computed by means of a surveyed river cross section and longitudinal slope.

This method, too, however, cannot always be applied in practice. There are, for instance, streams, particularly small ones, where observation of water stages had not been made and where no data on highest water levels are available. The cross sections and slopes of these rivers have also remained unmeasured.

In such cases, maximum discharges are determined by an appropriate conversion of the known maximum discharges of the adjoining cross sections of the same river, or of the nearest rivers with a similar hydrological regimen.

If the application of the latter method is also impossible, empirical correlations are used, derived by observations and measurements taken on various rivers together with materials supplied by the meteorological observation stations.

The formulas thus derived yield close values because the determination of the influence of all the factors on the formulation of discharge is impossible.

The factors most often influencing the magnitude of the integral flow are geographical while the formation of the maximum runoff per second principally depends on local influences.

The greatest influence is exerted on the magnitude of the entire runoff by the general quantity of liquid and solid precipitation and the distribution of such over a year, air temperature, saturation deficit, terrain configuration, ground permeability, retentive ability of the catchment basin, geological structure, number of lakes, marshes and forests, etc.

The factors influencing the value of the discharge per second may be divided into three groups. Meteorological phenomena come in the first group, geometrical characteristics of a catchment basin in the second, and physical characteristics of a river and catchment basin in the third.

The intensity, duration and size of the area of the simultaneous occurrence of rains or thawing snow belong to the first of these groups.

The size and shape of a catchment basin, type of tributaries, length of a river, etc. belong to the second group.

The physical characteristics of a river or its basin depend on the longitudinal and transverse section, density of the hydrographical network, geological structure of a catchment basin, losses involved in retention (endorheic drainage),

losses depending on the type of soil and vegetal cover, number of lakes, marshes and forests in a catchment basin, etc.

Among these factors, there are many the influence of which on the formation of runoff is comprehensible, but is as yet incapable of being expressed in a quantitative manner. For instance, the influence of the underground feeding of water to rivers is impossible to determine at the present stage of hydrological knowledge. It is only an approximate influence of such factors as quantity and intensity of precipitation, channel overgrowth by vegetation, terrain configuration, etc. that we are able to determine.

Theoretical hydrology tends, however, to derive general correlations which could facilitate hydrological computations even when direct observations are unavailable and data cannot be obtained by any indirect method.

There are in principle three different methods of deriving empirical formulas for computing maximum discharges.

The first of these consists in taking into account the intensity of rains subsequently converted into discharges simultaneously reducing the possible losses.

The formulas thus derived were at first very primitive. Subsequently, in deriving them the phenomenon of forming the runoff was analyzed. The analysis was connected with several very approximate assumptions. It was assumed, for instance, that the intensity of rain in time and space is unchangeable, that the water runoff along the slopes takes place in the form of a continuous layer, that the catchment basin has the shape of two inclined planes at the juncture of which the stream channel is located, etc.

The application of these and similar simplifications which clearly depart from the real conditions, causes formulas derived on the basis of the intensity of rains to be very approximate and to yield results which cannot be called satisfactory.

However, the formulas thus derived have been used for computing discharges when designing bridges and culverts on streams with small catchment basins (20 to 60 sq km). This was probably because formulas of this kind have been in general use for computing municipal sewer networks. In such a case, the duration and intensity of rain are determined on the basis of the time necessary for the water to flow to the cross section under study. On the other hand, the water losses are taken into account by introducing various coefficients of flow which under municipal conditions may be determined much more easily and accurately than for natural catchment areas.

The second method of deriving empirical formulas for computing maximum discharges consists in the analysis and generalization of the magnitudes of the maximum discharges observed. In this event, we study only the total influence of various factors on the formation of maximum discharges. The influence of these factors is expressed by introducing into a formula appropriate parameters which only indirectly take into account the character of rains.

Formulas of this type can be applied to compute maximum discharges in catchment basins both large and small.

The third method of deriving such formulas consists in taking into account direct characteristics of rains, together with maximum discharges observed and generalized characteristics of catchment basins.

Of all the three methods mentioned above for deriving empirical formulas, the first is the least adequate, because it is based on many closely approximate assumptions. Moreover, the number of observations of heavy rains necessary for drawing concrete conclusions has hitherto been insufficient.

2. Methods of Computing Maximum Discharge from Measurements

As already indicated discharge is the quantity of water flowing through a cross section in a unit of time, or the product of the cross section of a stream and the mean velocity of water in such cross section:

$$Q = Pv \text{ cu m/sec}$$

To explain the notion of discharge per second, let us imagine water particles, which all simultaneously began their movement in a stream cross section, and were suddenly stopped after one second. In that case, the water which flowed during one second will form a solid (Fig. 65) limited at the top by the water surface, at the bottom by the river bed, at one side by the cross section of the river, and at another side by the surface of velocity with the correlation:

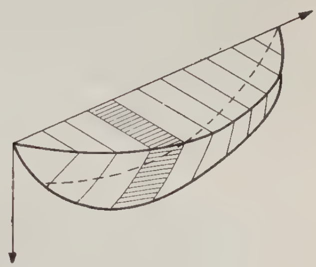


Fig. 65. Dischage per unit of time schematically presented

$$v = f(h, b)$$

where:

h — depth of the cross section,
b — width of the cross section.

This solid, also called the solid of discharge, will have a volume equal to the discharge per second.

Computing Maximum Discharges on the Basis of the Discharge Rating Curves

The discharge rating curves are prepared for some cross sections on rivers by means of measurements taken directly in the field at various water stages, from the highest to the lowest occurring during the period of observations. Then, the spans of bridges and openings of culverts are computed in relation to the discharge read on the discharge curve at the highest water stage.

This method should be considered most appropriate for determining maximum discharges with an unknown probability of occurrence, the more so if the curve of volume is drawn on the basis of observations over a sufficiently long period.

Computing Maximum Discharges on the Basis of Cross Section and Mean Velocity

The discharge curves elaborated on the basis of direct measurements were prepared for a very small number of cross sections. Generally, the mean velocity in a vertical at the highest water stage should be computed by empirical formulas.

All empirical formulas for computing velocities can be divided into two groups. The influence of the roughness of the bottom is covered by formulas of the first group, while the coefficient characterizing the roughness and physical features of a channel is not applied in formulas of the second group.

The first group of formulas does not enable direct velocity measurement; it serves only for the computation of the coefficient of resistance C, included in the well-known Chezy formula for uniform movement:

$$v = C\sqrt{Ri}$$

The formulas of the first group were derived in principle for computing pipes and artificial open channels (canals); at present, however, they are also applied for computation of natural streams. Since the correct choice of coefficients characterizing the physical properties of a channel is extremely difficult, velocities in natural streams computed by these formulas may deviate considerably from the real values.

The direct computation of the mean velocity of discharge, without consideration of the coefficients characterizing the roughness of the bottom, is facilitated by the second group of formulas. Such procedure is admissible in computing natural streams because the roughness of the bottom under the established river regimen constitutes a certain function of slopes, depth and velocity and is not a constant value as in the case of artificial channels. It is also known that a river scours and shapes its channel itself up to the moment when the relative balance is established.

It is obvious that a perfect balance cannot be achieved; the existence of such a balance may, however, be assumed approximately for our geological period. The formulas of this type are very convenient in use, since they eliminate the uncertainty existing in establishing coefficients of roughness in the formulas of the first group.

The formulas of the second group usually have the following form:

$$v - a\,t^n i^m$$

where:

a — numerical coefficient,
t — depth,
i — slope,
m, n — power exponents.

The empirical formulas for computing velocities of discharge will be discussed in detail, because discharges computed by these formulas facilitate the drawing (extrapolation) of upper sectors of the discharge curve for the high-water stages, for which direct velocity measurements are usually unavailable. The discharge curves, on the other hand, are necessary for establishing maximum discharges with unknown probability of occurrence, as well as for determining the maximum annual discharges, comprising the distributive series necessary for computing discharges with defined frequency.

Empirical Formulas of the First Group

The formulas of this group serve for computing the coefficient C for the following Chézy formula:

$$v = C\sqrt{Ri}$$

where:

R — hydraulic radius in m, equal to the ratio of the area of the cross section of the channel to the wetted perimeter,
i — slope of water surface expressed by a decimal fraction,
C — coefficient.

Many formulas serve for computing coefficient C, the usefulness of which was first delineated by Chézy in 1775, when it was planned to use the River Ivette to supply Paris with water. The coefficient C for computing the velocity of waters flowing in open channels was then established by Chézy by means of measurements in the Courpalet ditches:

$$C = 31$$

During the subsequent period, another value, i. e.:

$$C = 17$$

for the discharge in the conduits of the city of Rennes was advanced by Chézy.

It appears that even then Chézy understood the instability of the coefficient C.

The Ganguillet-Kutter (1869) formula has the following form:

$$C = \frac{23 + \frac{1}{n} + \frac{0.00155}{i}}{1 + \left(23 + \frac{0.00155}{i}\right)\frac{n}{\sqrt{R}}}$$

where:

n — coefficient of roughness,

R, i — as in the previously shown Chézy formula.

The value of the coefficient n should be taken from Tables 16 and 19 for natural streams.

Satisfactory results are given by the Ganguillet-Kutter formula, in which all the three values (R, i, n) influencing the movement of water are taken into account.

The Manning formula (1890, shortened) has the following form:

$$C = \frac{1}{n} R^{\frac{1}{6}}$$

The values of the coefficient n are accepted according to Table 16.

The Forchheimer (1923) formula:

$$C = \frac{1}{n} R^{\frac{1}{5}}$$

The values of the coefficient n are accepted according to Table 16.

The Bazin formula (of 1897, new):

$$C = \frac{87\sqrt{R}}{\gamma + \sqrt{R}}$$

The values of the coefficient γ are presented in Tables 17 and 19.

Pavlovskii formula (1925)

Pavlovskii formula, which can be used at a hydraulic radius equal to $0.1 \leqslant R \leqslant 3.0$ m, has the following form:

$$C = \frac{1}{n} R^{a}$$

where:

R — hydraulic radius,

n — coefficient depending on the roughness of the bottom and banks of a channel,

a — power exponent depending on R and n, assumed according to the formula $a = 2.5\sqrt{n} - 0.13 - 0.75\sqrt{R}\,(\sqrt{n} - 0.10)$. The values of the exponent a for various n and R are presented in Table 18.

The shortened Pavlovskii formula:

$$C = \frac{1}{n} R^{a}$$

where:

$$a = 1.5\sqrt{n} \text{ for } 0.1 \leqslant R \leqslant 1$$

$$a = 1.3\sqrt{n} \text{ for } 1 < R \leqslant 3$$

Coefficients of roughness n for the Pavlovskii formulas are assumed according to Tables 16 and 19 prepared by Sribnyi on the basis of ample material.

Goncharov formula (1938):

$$C = \frac{19.6}{t^{0.2}} R^{0.2}$$

where:

R — hydraulic radius in m,

t — mean depth of roughness depressions in m (absolute roughness) presented in Table 19; the multiplier $\frac{19.6}{t^{0.2}}$ corresponds to the value $\frac{1}{n}$ in Table 19, and, therefore, it may also be assumed according to that table.

Agroskin formula (1948):

$$C = 17.72\,(K + \log R)$$

where:

R — hydraulic radius in m,

K — parameter of roughness, which can be adopted from Tables 16 and 19 or computed from the formula:

$$K = \frac{0.0564}{n}$$

The value of the coefficient of resistance C, determined on the basis of formulas belonging to the first group, serves for computing the mean velocity by the Chézy formula referred to above.

Computing the Discharge

Discharge is computed by multiplying the mean velocity thus obtained by the cross section of the stream.

The maximum discharge is usually computed as a sum of discharges for the main channel and valley flat because these parts of the cross section usually have differing degrees of roughness:

$$Q = Q_k + Q_n + Q_1$$

Consequently the following data should be available for computing the discharge in a river or canal at the highest (or any) water stage:

(a) area of the cross section,

Table 16

Values of the Coefficients of Roughness

No.	Kind of bottom and walls of a channel	Values		
		n	$\frac{1}{n}$	K
1	Enamelled surfaces. Boards very carefully planed and fitted.	0.009	111.1	6.26
2	Planed boards. Pure cement plaster	0.010	100.0	5.64
3	Cement plasters (mixed with 1/3 sand). New, steel cast-iron and stoneware pipes, carefully laid and joined.	0.011	90.9	5.13
4	Unplaned boards, well filled. Very carefully laid concrete. Waterwork pipes under normal conditions. Very clean sewer pipes.	0.012	83.3	4.70
5	Stone block wall. Very carefully built brick wall. Unplaned boards, not precisely fitted. Slightly soiled water pipes. Sewer pipes under normal conditions.	0.013	76.9	4.34
6	Soiled pipes. Concrete canals under normal conditions. Wall made of average quality bricks.	0.014	71.4	4.03
7	Carelessly built brick walls. Walls made of broken stones with well fitted stone edges.	0.015	66.7	3.76
8	Ordinary wall made of broken stone in a satisfactory condition. Old brick walls. Concretes not carefully laid. Very smooth rocks.	0.017	58.8	3.32
9	Canals covered with a thick layer of stable loam. Canals dug in compact loess or fine gravel covered with a thin layer of loam, well kept.	0.018	55.6	3.13
10	Carelessly built broken stone walls. Dry walls of coarse stones. Cobbled area. Canals carefully hewn in a rock. Canals dug in loess, compact gravel or earth, covered with a thin layer of loam, fairly well kept.	0.020	50.0	2.82
11	Area cobbled with large stones with sharp edges. Canals dug in compact clay and rock with a carelessly evened surface. Large canals dug in earth, fairly well kept.	0.0225	44.4	2.50

(Table 16 continued)

No.	Kind of bottom and walls of a channel	Values		
		n	$\frac{1}{n}$	K
12	Large canals dug in earth, fairly well preserved; and small canals well kept. Rivers and streams under good conditions (without impurities and abundant vegetal cover).	0.025	40.0	2.26
13	Canals dug in earth: large — under conditions below average, small — under average conditions.	0.0275	36.4	2.05
14	Canals under comparatively bad conditions (in some place overgrown by grass, many stones, partially land slipped banks).	0.030	33.3	1.88
15	Canals under very bad conditions, with variable cross sections, great quantity of stones, overgrown by vegetation.	0.035	28.6	1.61
16	Canals under exceptionally bad conditions (large stones and fragments of rocks in the channel, overgrown by sweet flag, many thick roots, etc.).	0.040	25.0	1.41

Table 17

Values of the Coefficient γ for the Bazin Formula

No.	Kind of channel walls	γ
1	Very smooth walls: planed boards, smooth cement plaster, etc.	0.06
2	Smooth walls: unplaned boards, walls of bricks and stone block. Concrete and cast-iron pipes.	0.16
3	Walls made of broken stone and fairly smooth concrete.	0.46
4	Walls formed by an uneven concrete wall, made of a very rough concrete, cobbled.	0.85
5	Earth embankments, normally kept.	1.30
6	Earth channels offering great resistance because of their overgrowth or deposited material consisting of boulders.	1.75

(b) hydraulic radius corresponding with this area,

(c) longitudinal slope of the water surface,

(d) data on the condition of the channel necessary for the correct choice of coefficient C.

Table 18

Values of the Coefficient α in the Pavlovskii Formula $C = \frac{1}{n} R^{\alpha}$

n \ R	0.10	0.20	0.30	0.40	0.80	1.00	1.50	2.0	2.50	3.0
0.010	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120
0.013	0.152	0.151	0.149	0.147	0.146	0.145	0.141	0.140	0.139	0.137
0.015	0.171	0.168	0.165	0.163	0.161	0.159	0.155	0.152	0.150	0.147
0.017	0.189	0.186	0.182	0.179	0.176	0.173	0.168	0.164	0.160	0.156
0.020	0.214	0.210	0.204	0.200	0.196	0.193	0.184	0.178	0.174	0.169
0.0225	0.235	0.228	0.221	0.216	0.211	0.207	0.199	0.192	0.185	0.180
0.025	0.251	0.246	0.238	0.232	0.227	0.222	0.212	0.204	0.197	0.190
0.0275	0.269	0.263	0.254	0.247	0.241	0.235	0.224	0.215	0.206	0.199
0.030	0.286	0.279	0.268	0.261	0.254	0.248	0.238	0.225	0.216	0.208
0.0325	0.305	0.296	0.283	0.275	0.268	0.261	0.247	0.236	0.226	0.217
0.035	0.318	0.309	0.297	0.288	0.280	0.273	0.258	0.246	0.235	0.225
0.0375	0.332	0.323	0.309	0.299	0.291	0.284	0.268	0.255	0.243	0.232
0.040	0.346	0.337	0.323	0.312	0.303	0.295	0.278	0.264	0.251	0.240

All the data given above are obtained by direct measurements and field studies and the magnitude of coefficient C can be computed by the formulas already given.

The formulas with constant exponent of power for the value R (of the Manning or Forchheimer type) are convenient in computations and, therefore, recommended for practical use. The following values of the exponent should be applied:

$\frac{1}{6}$ in computing channels with slight roughness

$$(0.010 \leqslant n \leqslant 0.015),$$

$\frac{1}{5}$ in computing channels with average roughness

$$(0.015 < n \leqslant 0.025),$$

$\frac{1}{4}$ in computing channels with great roughness

$$(n > 0.025).$$

Table 19

Values of the Coefficients of Roughness n and γ, and the Parameters of Roughness K Corresponding with Them

No.	Channel characteristics	Values			
		n	$\frac{1}{n}$	γ	K
1	Completely even and clean natural streams, straight in plan.	0.025	40	1.25	2.256
2	Channels of rivers, mostly large and average, of the lowland type. State of river bed — good, favorable conditions of water discharge.	0.033	30	1.75	1.692
3	Relatively clean channels of permanent lowland streams with usual conditions, meandering with insignificant deviations of the direction of single water streams, or rectilinear, but with small bottom unevennesses (scours, stones). Regularly, well-shaped channels of mountain rivers in their lower reaches. Earth channels of drying streams, well kept.	0.040	25	2.75	1.410
4	Channels of large and average rivers, average soiled, meandering and partially overgrown by vegetation, stony and with turbulent water movement. Drying streams, during highwater stages carrying considerable quantities of sediment, with overgrown channel or stream with boulders. Valley flats of large and average rivers, fairly well developed, covered with vegetation (grass, bushes).	0.050	20	3.75	1.128
5	Channels of drying streams, markedly soiled and meandering. Considerably overgrown, poorly developed valley flats. Sector of the lowland rivers with water jumps; mountain channels formed by boulders, with an uneven water level.	0.067	15	5.50	0.846
6	Rivers and valley flats greatly overgrown with deep scour. Mountain channels formed by boulders, with turbulent water movement and very uneven water level.	0.080	12.5	7.00	0.705
7	Valley flats — as above, but with a very unsteady current and water streams taking an oblique direction. Mountain channels of waterfall type, distinct water jumps and meandering channel. Rising water losing its transparency and acquiring a white color; the noise of the stream prevents hearing speach.	0.100	10	9.00	0.564

(Table 19 continued)

No.	Channel characteristics	Values			
		n	$\frac{1}{n}$	γ	K
8	Marshy rivers, overgrown, water almost still. Valley flats covered with woods; water does not flow off some depressions.	0.133	7.5	12.0	0.423
9	Marshy, stony streams, etc. Valley flats completely covered with woods.	0.200	5	20.0	0.282
10	Catchment basin slopes in a natural state $\left(\frac{1}{n}\right.$ may amount to between 1 and 4 depending on the character of slopes$\left.\right)$.	0.400	2.5	—	0.141

Tables 21, 22, 23, 24 and 25 are prepared in order to facilitate the computation of coefficient C for the Chézy formula.

Empirical Formulas of the Second Group

A number of empirical formulas without coefficients of roughness have been derived, which facilitate direct computing of the mean velocity in the cross section of a stream.

The best known among these formulas are those worked out by Matakiewicz, Sokolovskii-Pokrovskii, Groeger, Hermanek, Christen, Siedek and Lindboe. Most often used for computing natural channels are the following formulas of this type:

Matakiewicz formula (1931):

$$v = 35.4\, t^{0.7}\, i^{0.49+10\, i}$$

where:

t — mean depth in m, computed from the equation

$$t = \frac{F}{B}$$

i — slope of water surface expressed as a decimal fraction,
F — area of the cross section of the stream in sq m,
b — width of the water surface in m.

Table 20

Coefficients of Roughness According to Gonacharov

Category	Character of channel	Dimensions of the roughness depressions t, in m	n
I	Very smooth surfaces of metal or well fitted planed boards. Cement plaster very carefully burned.	0.0005	0.011
II	Even asphalted surfaces of black metals, or timber. Planed boards. Burned cement plaster average maintained.	0.0005 – 0.001	0.011 – 0.013
III	Surfaces of black metals under normal conditions. Wooden troughs made of boards planed transversely. Cement plaster unburned. Troughs of stone blocks or bricks, well or average maintained.	0.001 – 0.002	0.013 – 0.015
IV	Wooden troughs and pipes. Concrete troughs and pipes with rough surface. Well constructed flagstone walls.	0.002 – 0.005	0.015 – 0.018
V	Rectilinear canals dug in clay, covered with a layer of loam, well kept. Even sandy channels. Rough-surfaced concrete canals. Usual stone walls. Old walls made of bricks. Smooth rocks.	0.005 – 0.01	0.018 – 0.02
VI	Even earth channels without major depressions. Gravel channels with stones up to 50 mm in diameter. Primitively built concrete and stone surfaces. Carefully cobbled area. Canals accurately hewn out of rock.	0.010 – 0.020	0.02 – 0.023
VII	Channels formed by boulders. Earth and gravel channels with bottoms strewn with stones. Plowed and harrowed surface of ground. Ordinary canals hewn out of rock. Average cobbled area.	0.020 – 0.040	0.023 – 0.027
VIII	Gravel and boulder channels in bad condition. Scoured earthen channels. Old cobbles. Walls of broken stone, stones with protruding edges.	0.040 – 0.080	0.027 – 0.031

Table 21

Values of the Coefficientt C according to the Bazin Formula

R \ γ	0.06	0.16	0.46	0.85	1.30	1.75	R \ γ	0.06	0.16	0.46	0.85	1.30	1.75
0.05	68,5	50.7	28.4	18.1	12.8	9.9	0.45	79.8	70.2	51.6	38.4	29.6	24.1
0.06	69.8	52.6	30.2	19.4	13.8	10.7	0.46	79.9	70.4	51.8	38.6	29.8	24.3
0.07	70.9	54.2	31.7	20.6	14.7	11.4	0.47	80.0	70.5	52.0	38.8	30.0	24.5
0.08	71.8	55.6	33.1	21.7	15.5	12.1	0.48	80.0	70.6	52.3	39.1	30.2	24.7
0.09	72.5	56.7	34.4	22.7	16.3	12.7	0.49	80.1	70.8	52.5	39.3	30.4	24.8
0.10	73.1	57.7	35.5	23.6	17.0	13.3	0.50	80.2	70.9	52.7	39.5	30.6	25.0
0.11	73.6	58.7	36.5	24.4	17.7	13.9	0.55	80.4	71.5	53.7	40.5	31.6	25.9
0.12	74.1	59.5	37.4	25.2	18.3	14.4	0.60	80.7	72.1	54.6	41.4	32.5	26.7
0.13	74.6	60.2	38.2	25.9	18.9	14.9	0.65	80.9	72.6	55.4	42.3	33.3	27.4
0.14	75.0	60.9	39.0	26.7	19.4	15.3	0.70	81.1	73.0	56.1	43.1	34.1	28.1
0.15	75.3	61.5	39.7	27.2	19.9	15.8	0.75	81.3	73.4	56.8	43.9	34.8	28.8
0.16	75.6	62.1	40.5	27.8	20.4	16.2	0.80	81.5	73.8	57.4	44.6	35.5	29.4
0.17	75.9	62.7	41.2	28.4	20.9	16.6	0.85	81.7	74.1	58.0	45.2	36.1	30.0
0.18	76.2	63.2	41.8	29.0	21.4	17.0	0.90	81.8	74.4	58.6	45.9	36.5	30.6
0.19	76.5	63.6	42.4	29.5	21.8	17.3	0.95	81.9	74.7	59.1	46.5	37.3	31.1
0.20	76.7	64.1	42.9	30.0	22.3	17.7	1.00	82.0	75.0	59.6	47.0	37.8	31.6
0.21	76.9	64.5	43.5	30.5	22.7	18.1	1.10	82.2	75.4	60.5	48.0	38.8	32.6
0.22	77.1	64.9	44.0	30.9	23.1	18.4	1.20	82.4	75.9	61.3	48.9	39.7	33.5
0.23	77.3	65.2	44.4	31.4	23.4	18.7	1.30	82.6	76.3	62.0	49.8	40.6	34.3
0.24	77.5	65.5	44.8	31.8	23.8	19.0	1.40	82.8	76.6	62.6	50.6	41.4	35.1

0.25	77.6	65.9	45.3	32.2	24.2	19.3	1.50	82.9	76.9	63.2	51.3	42.2	35.8
0.26	77.8	66.2	45.7	32.6	24.5	19.6	1.60	83.0	77.2	63.8	52.0	42.9	36.5
0.27	78.0	66.5	46.1	33.0	24.8	19.9	1.70	83.1	77.5	64.3	52.6	43.6	37.1
0.28	78.1	66.8	46.5	33.4	25.2	20.2	1.80	83.2	77.7	64.8	53.2	44.2	37.7
0.29	78.3	67.0	46.9	33.7	25.5	20.5	1.90	83.3	77.9	65.2	53.8	44.8	38.3
0.30	78.4	67.3	47.3	34.1	25.8	20.7	2.00	83.4	78.1	65.6	54.3	45.3	38.9
0.31	78.5	67.6	47.6	34.3	26.1	21.0	2.20	83.6	78.5	66.4	55.3	46.4	39.9
0.32	78.6	67.8	47.9	34.7	26.4	21.2	2.40	83.7	78.8	67.1	56.2	47.3	40.8
0.33	78.8	68.0	48.2	35.1	26.7	21.5	2.60	83.8	79.1	67.7	57.0	48.1	41.7
0.34	78.9	68.2	48.5	35.4	26.9	21.7	2.80	83.9	79.4	68.2	57.7	48.9	42.5
0.35	79.0	68.4	48.8	35.7	27.2	22.0	3.00	84.0	79.6	68.7	58.3	49.7	43.3
0.36	79.1	68.6	49.2	36.0	27.5	22.2	3.20	84.1	79.8	69.2	58.9	50.4	44.0
0.37	79.2	68.8	49.5	36.3	27.7	22.4	3.40	84.2	80.0	69.6	59.5	51.0	44.6
0.38	79.2	69.0	49.8	36.6	28.0	22.7	3.60	84.3	80.2	70.0	60.1	51.6	45.2
0.39	79.3	69.2	50.1	36.8	28.2	22.9	3.80	84.4	80.4	70.4	60.6	52.2	45.8
0.40	79.4	69.4	50.4	37.1	28.5	23.1	4.00	84.4	80.5	70.7	61.0	52.7	46.4
0.41	79.5	69.6	50.6	37.4	28.7	23.3	4.50	84.6	80.9	71.5	62.1	53.9	47.6
0.42	79.6	69.7	50.9	37.6	28.9	23.5	5.00	84.7	81.2	72.1	63.0	55.0	48.8
0.43	79.7	69.9	51.1	37.9	29.2	23.7	5.50	84.8	81.4	72.7	63.8	56.0	49.8
0.44	79.7	70.1	51.4	38.1	29.4	23.9	6.00	84.9	81.6	73.2	64.6	56.6	50.7

Table 22

Values of the Coefficient C for the Manning Formula $C = \frac{1}{n} R^{\frac{1}{6}}$

R \ n	0.011	0.013	0.014	0.017	0.020	0.025	0.030	0.035	0.040
0.30	74.4	63.0	58.4	48.1	40.9	32.7	27.3	23.4	20.4
0.32	75.2	63.6	59.1	48.6	41.4	33.1	27.5	23.6	20.7
0.34	76.0	64.3	59.7	49.1	41.8	33.4	27.8	23.9	20.9
0.36	76.7	64.9	60.3	49.6	42.2	33.7	28.1	24.1	21.1
0.38	77.4	65.5	60.8	50.1	42.6	34.0	28.4	24.3	21.3
0.40	78.1	66.0	61.3	50.5	42.9	34.3	28.6	24.5	21.4
0.42	78.7	66.6	61.8	50.9	43.3	34.6	28.9	24.7	21.6
0.44	79.3	67.1	62.3	51.3	43.6	34.9	29.1	24.9	21.8
0.46	79.9	67.6	62.8	51.7	43.9	35.2	29.3	25.1	22.0
0.48	80.4	68.1	63.2	52.0	44.2	35.4	29.5	25.3	22.1
0.50	81.0	68.5	63.6	52.4	44.5	35.6	29.7	25.5	22.3
0.55	82.3	69.6	64.6	53.3	45.3	36.2	30.2	25.9	22.6
0.60	83.5	70.6	65.6	54.0	45.9	36.7	30.6	26.2	23.0
0.65	84.6	71.6	66.5	54.7	46.5	37.2	31.0	26.6	23.3
0.70	85.7	72.5	67.3	55.4	47.1	37.7	31.4	26.9	23.6
0.75	86.7	73.3	68.1	56.1	47.7	38.1	31.8	27.2	23.8
0.80	87.6	74.1	68.8	56.8	48.2	38.5	32.1	27.5	24.1

0.85	88.5	74.9	69.5	57.2	48.7	38.9	32.4	27.8	24.3
0.90	89.3	75.6	70.2	57.8	49.1	39.3	32.8	28.1	24.6
0.95	90.1	76.3	70.8	58.3	49.6	39.7	33.0	28.3	24.8
1.00	90.9	77.0	71.4	58.8	50.0	40.0	33.3	28.6	25.0
1.10	92.4	78.2	72.6	59.8	50.8	40.6	33.9	29.0	25.4
1.20	93.7	79.3	73.6	60.6	51.5	41.2	34.4	29.5	25.8
1.30	95.0	80.4	74.6	61.5	52.2	41.8	34.8	29.8	26.1
1.40	96.2	81.4	75.6	62.2	52.9	42.3	35.3	30.2	26.4
1.50	97.3	82.3	76.4	62.9	53.5	42.8	35.7	30.6	26.8
1.60	98.3	83.2	77.2	63.6	54.1	43.3	36.1	30.9	27.0
1.70	99.3	84.1	78.0	64.3	54.6	43.7	36.4	31.2	27.3
1.80	100.3	84.8	78.8	64.9	55.1	44.1	36.8	31.5	27.6
1.90	101.2	85.6	79.5	65.5	55.6	44.5	37.1	31.8	27.8
2.00	102.0	86.3	80.2	66.0	56.1	44.9	37.4	32.1	28.1
2.20	103.7	87.7	81.5	67.1	57.0	45.6	38.0	32.6	28.5
2.40	105.2	89.0	82.7	68.1	57.8	46.3	38.6	33.1	28.9
2.60	106.6	90.2	83.8	69.0	58.6	46.9	39.1	33.5	29.3
2.80	108.0	91.3	84.8	69.8	59.4	47.5	39.6	33.9	29.7
3.00	109.2	92.4	85.8	70.6	60.0	48.0	40.0	34.3	30.0
3.50	112.0	94.8	88.0	72.5	61.6	49.3	41.1	35.2	30.8
4.00	114.5	97.0	90.0	74.1	63.0	50.4	42.0	36.0	31.5
4.50	116.8	98.8	91.8	75.6	64.2	51.4	42.8	36.7	32.1
5.00	118.9	100.6	93.4	76.9	65.4	52.3	43.6	37.4	32.7

Table 23

Values of the Coefficient C for the Forchheimer Formula $C = \frac{1}{n} R^{\frac{1}{5}}$

R \ n	0.011	0.013	0.014	0.017	0.020	0.025	0.030	0.035	0.040
0.30	71.5	60.5	56.1	46.2	39.3	31.4	26.2	22.5	19.7
0.32	72.4	61.3	56.9	46.8	39.8	31.9	26.5	22.8	19.9
0.34	73.3	62.0	57.6	47.4	40.3	32.2	26.9	23.0	20.2
0.36	74.1	62.7	58.2	48.0	40.8	32.6	27.2	23.3	20.4
0.38	74.9	63.4	58.9	48.5	41.2	33.0	27.5	23.6	20.6
0.40	75.7	64.1	59.5	49.0	41.6	33.2	27.7	23.8	20.8
0.42	76.4	64.7	60.1	49.5	42.0	33.6	28.0	24.0	21.0
0.44	77.2	65.3	60.6	49.9	42.4	33.9	28.3	24.3	21.2
0.46	77.8	65.9	61.2	50.4	42.8	34.3	28.5	24.5	21.4
0.48	78.5	66.4	61.7	50.8	43.2	34.5	28.8	24.7	21.6
0.50	79.2	67.0	62.2	51.2	43.5	34.8	29.0	24.9	21.8
0.55	80.7	68.3	63.4	52.2	44.4	35.5	29.6	25.4	22.2
0.60	82.1	69.5	64.5	53.1	45.2	36.1	30.1	25.8	22.6
0.65	83.4	70.6	65.5	54.0	45.9	36.7	30.6	26.2	22.9
0.70	84.7	71.6	66.5	54.8	46.6	37.3	31.0	26.6	23.3
0.75	85.8	72.6	67.4	55.5	47.2	37.8	31.5	27.0	23.6
0.80	87.0	73.6	68.3	56.3	47.8	38.3	31.9	27.3	23.9

0.85	88.0	74.5	69.1	56.9	48.4	38.7	32.3	27.7	24.2
0.90	89.0	75.3	69.9	57.6	49.0	39.2	32.6	28.0	24.5
0.95	90.0	76.1	70.7	58.2	49.5	39.6	33.0	28.3	24.8
1.00	90.9	76.9	71.4	58.8	50.0	40.0	33.3	28.6	25.0
1.10	92.7	78.4	72.8	60.0	51.0	40.8	34.0	29.1	25.5
1.20	94.3	79.8	74.1	61.0	51.9	41.5	34.6	29.6	25.9
1.30	95.8	81.1	75.3	62.0	52.7	42.2	35.1	30.1	26.4
1.40	97.2	82.3	76.4	62.9	53.5	42.8	35.7	30.6	26.7
1.50	98.6	83.4	77.5	63.8	54.2	43.4	36.2	31.0	27.1
1.60	99.9	84.4	78.5	64.6	55.0	43.9	36.6	31.4	27.5
1.70	101.1	85.5	79.4	65.4	55.6	44.5	37.1	31.8	27.8
1.80	102.3	86.5	80.4	66.2	56.2	45.0	37.5	32.1	28.1
1.90	103.4	87.5	81.2	66.9	56.9	45.5	37.9	32.5	28.4
2.00	104.4	88.4	82.1	67.5	57.4	46.0	38.3	32.8	28.7
2.20	106.4	90.1	83.6	68.9	58.5	46.8	39.0	33.5	29.3
2.40	108.3	91.7	85.1	70.1	59.6	47.7	39.7	34.0	29.8
2.60	110.1	93.1	86.5	71.2	60.5	48.4	40.4	34.6	30.3
2.80	111.7	94.5	87.8	72.3	61.4	49.2	41.0	35.1	30.7
3.00	113.3	95.8	89.0	73.3	62.3	49.8	41.5	35.6	31.1
3.50	116.8	98.8	91.8	75.6	64.2	51.4	42.8	36.7	32.1
4.00	119.9	101.5	94.3	77.6	66.0	52.8	44.0	37.7	33.0
4.50	122.8	103.9	96.5	79.7	67.6	54.0	45.0	38.6	33.8
5.00	125.4	106.1	98.6	81.2	69.0	55.2	46.0	39.4	34.5

Table 24

Values of the Coefficient C for the Formula $C = \frac{1}{n} R^{\frac{1}{4}}$

R \ n	0.011	0.013	0.014	0.017	0.020	0.025	0.030	0.035	0.040
0.30	67.281	56.931	52.864	43.535	37.005	29.604	24.670	21.145	18.502
0.32	68.373	57.854	53.721	44.241	37.605	30.084	25.070	21.488	18.802
0.34	69.418	58.738	54.542	44.917	38.180	30.544	25.453	21.816	19.090
0.36	70.418	59.584	55.328	45.564	38.730	30.984	25.820	22.131	19.365
0.38	71.373	60.392	56.078	46.182	39.255	31.404	26.170	22.431	19.627
0.40	72.230	61.177	56.807	46.782	39.765	31.812	26.510	22.722	19.882
0.42	73.182	61.923	57.499	47.352	40.250	32.200	26.833	22.999	20.125
0.44	74.036	62.646	58.171	47.905	40.720	32.576	27.146	23.268	20.360
0.46	74.863	63.346	58.821	48.441	41.175	32.940	27.450	23.528	20.587
0.48	75.673	64.031	59.457	48.964	41.620	33.296	27.746	23.782	20.810
0.50	76.445	64.684	60.064	49.464	42.045	33.636	28.030	24.025	21.022
0.55	78.291	66.246	61.514	50.658	43.060	34.448	28.706	24.605	21.530
0.60	80.010	67.670	62.864	51.770	44.005	35.204	29.336	25.145	22.002
0.65	81.627	69.069	64.135	52.812	44.895	35.916	29.930	25.654	22.447
0.70	83.145	70.354	65.328	53.799	45.730	36.584	30.486	26.131	22.865
0.75	84.600	71.584	66.471	54.740	46.530	37.224	31.020	26.588	23.265
0.80	85.973	72.746	67.549	55.629	47.285	37.828	31.523	27.019	23.642

0.85	87.354	73.915	68.635	56.523	48.045	38.436	32.030	27.454	24.022
0.90	88.545	74.923	69.570	57.294	48.700	38.960	32.466	27.828	24.350
0.95	89.745	75.938	70.514	58.070	49.360	39.488	32.906	28.205	24.680
1.00	90.909	76.923	71.428	58.823	50.000	40.000	33.333	28.571	25.000
1.10	93.091	78.769	73.142	60.234	51.200	40.960	34.132	29.257	25.600
1.20	95.091	80.461	74.714	61.529	52.300	41.840	34.866	29.885	26.150
1.30	97.091	82.154	76.285	62.823	53.400	42.720	35.600	30.514	26.700
1.40	98.909	83.692	77.714	63.999	54.400	43.520	36.266	31.085	27.200
1.50	100.636	85.154	79.071	65.117	55.350	44.280	36.900	31.628	27.675
1.60	102.272	86.538	80.356	66.176	56.250	45.000	37.450	32.142	28.125
1.70	103.820	87.846	81.571	67.176	57.100	45.680	38.066	32.628	28.550
1.80	105.363	89.154	82.785	68.176	57.950	46.360	38.632	33.114	28.975
1.90	106.727	90.308	83.856	69.058	58.700	46.960	39.133	33.542	29.350
2.00	108.091	91.461	84.928	69.940	59.450	47.560	39.633	33.971	29.725
2.20	110.727	93.692	86.999	71.646	60.900	48.720	40.599	34.799	30.450
2.40	113.182	95.769	88.928	73.235	62.250	49.800	41.499	35.571	31.125
2.60	115.454	97.692	90.713	74.705	63.500	50.800	42.333	36.285	31.750
2.80	117.636	99.538	92.428	76.117	64.700	51.760	43.133	36.971	32.350
3.00	119.636	101.230	93.999	77.411	65.800	52.640	43.866	37.599	32.900
3.50	124.333	105.231	97.714	80.471	68.400	54.720	45.600	39.086	34.200
4.00	128.545	108.769	100.999	83.176	70.700	56.560	47.133	40.399	35.350
4.50	132.454	112.080	104.070	85.705	72.850	58.280	48.566	41.628	36.425
5.00	135.818	114.922	106.713	87.881	74.700	59.760	49.799	42.685	37.350

Table 25

Values for the Coefficient C for the Pavlovski Formula $C = \frac{1}{n} R^a$

where $a = 2.5\sqrt{n} - 0.13 - 0.75\sqrt{R}\left(\sqrt{n} - 0.1\right)$

R \ n	0.011	0.013	0.017	0.020	0.025	0.030	0.035	0.040
0.05	61.3	48.7	33.2	26.1	18.6	13.9	10.9	8.9
0.06	62.8	50.1	34.4	27.2	19.5	14.7	11.5	9.3
0.07	64.1	51.3	35.5	28.2	20.4	15.5	12.2	9.9
0.08	65.2	52.4	36.4	29.0	21.1	16.1	12.8	10.3
0.10	67.2	54.3	38.1	30.6	22.4	17.3	13.8	11.2
0.12	68.8	55.8	39.5	31.6	23.5	18.3	14.7	12.1
0.14	70.3	57.2	40.7	33.0	24.5	19.1	15.4	12.8
0.16	71.5	58.4	41.8	34.0	25.4	19.9	16.1	13.4
0.18	72.6	59.5	42.7	34.8	26.2	20.6	16.8	14.0
0.20	73.7	60.4	43.6	35.7	26.9	21.3	17.4	14.5
0.22	74.6	61.3	44.4	36.4	27.6	21.9	17.9	15.0
0.24	75.5	62.1	45.2	37.1	28.3	22.5	18.5	15.5
0.26	76.3	62.9	45.9	37.8	28.8	23.0	18.9	16.0
0.28	77.0	63.6	46.5	38.4	29.4	23.5	19.4	16.4
0.30	77.7	64.3	47.2	39.0	29.9	24.0	19.9	16.8
0.35	79.3	65.8	48.6	40.3	31.1	25.1	20.9	17.8
0.40	80.7	67.1	49.8	41.5	32.2	26.0	21.8	18.6

0.45	82.0	68.4	50.9	42.5	33.1	26.9	22.6	19.4
0.50	83.1	69.5	51.9	43.5	34.0	27.8	23.4	20.1
0.55	84.1	70.4	52.8	44.4	34.8	28.5	24.0	20.7
0.60	85.3	71.4	53.7	45.2	35.5	29.2	24.7	21.3
0.65	86.0	72.2	54.5	45.9	36.2	29.8	25.3	21.9
0.70	86.8	73.0	55.2	46.6	36.9	30.4	25.8	22.4
0.80	88.3	74.5	56.5	47.9	38.0	31.5	26.8	23.4
0.90	89.4	75.5	57.5	48.8	38.9	32.3	27.6	24.1
1.00	90.9	76.9	58.5	50.0	40.0	33.3	28.6	25.0
1.10	92.0	78.0	59.8	50.9	40.9	34.1	29.3	25.7
1.20	93.1	79.0	60.7	51.8	41.6	34.8	30.0	26.3
1.30	94.0	79.9	61.5	52.5	42.3	35.5	30.6	26.9
1.50	95.7	81.5	62.9	53.9	43.6	36.7	31.7	28.0
1.70	97.3	82.9	64.3	55.1	44.7	37.7	32.7	28.9
2.00	99.3	84.8	65.9	56.6	46.0	38.9	33.8	30.0
2.50	102.1	87.3	68.1	58.7	47.9	40.6	35.4	31.5
3.00	104.4	89.4	69.8	60.3	49.3	41.9	36.6	32.5
4.00	108.1	92.6	72.5	62.5	51.2	43.6	38.1	33.9
5.00	111.0	95.1	74.2	64.1	52.4	44.6	38.9	34.6

Values of the Function of Slope $f(i)$ for the Matakiewicz Formula

Slope i	$f(i)$	Slope i	$f(i)$	Slope i	$f(i)$	Slope i	$f(i)$
0.000010	0.12	0.000051	0.26	0.000092	0.34	0.00042	0.71
11	0.12	52	0.26	93	0.34	43	0.72
12	0.13	53	0.26	94	0.35	44	0.73
13	0.13	54	0.26	95	0.35	45	0.73
14	0.14	55	0.27	96	0.35	46	0.74
15	0.14	56	0.27	97	0.35	47	0.75
16	0.14	57	0.27	98	0.36	48	0.76
17	0.15	58	0.27	99	0.36	49	0.76
18	0.15	59	0.28			0.00050	0.77
19	0.16	0.000060	0.28	0.00010	0.36	51	0.78
0.000020	0.16	61	0.28	11	0.37	52	0.78
21	0.17	62	0.28	12	0.39	53	0.79
22	0.17	63	0.28	13	0.41	54	0.80
23	0.18	64	0.29	14	0.42	55	0.80
24	0.18	65	0.29	15	0.44	56	0.81
25	0.18	66	0.29	16	0.45	57	0.81
26	0.19	67	0.29	17	0.47	58	0.82
27	0.19	68	0.30	18	0.48	59	0.83
28	0.19	69	0.30	19	0.49	0.00060	0.84
29	0.20	0.000070	0.30	0.00020	0.50	61	0.84
0.000030	0.20	71	0.30	21	0.51	62	0.85
31	0.20	72	0.30	22	0.52	63	0.85
32	0.21	73	0.31	23	0.54	64	0.86
33	0.21	74	0.31	24	0.55	65	0.86
34	0.21	75	0.31	25	0.56	66	0.87
35	0.22	76	0.31	26	0.57	67	0.88
36	0.22	77	0.31	27	0.58	68	0.88
37	0.22	78	0.31	28	0.59	69	0.89
38	0.23	79	0.32	29	0.60	0.00070	0.90
39	0.23	0.000080	0.32	0.00030	0.61	71	0.90
0.000040	0.23	81	0.32	31	0.62	72	0.91
41	0.23	82	0.32	32	0.63	73	0.92
42	0.24	83	0.33	33	0.64	74	0.92
43	0.24	84	0.33	34	0.64	75	0.93
44	0.24	85	0.33	35	0.65	76	0.93
45	0.24	86	0.33	36	0.66	77	0.94
46	0.25	87	0.33	37	0.67	78	0.94
47	0.25	88	0.34	38	0.68	79	0.95
48	0.25	89	0.34	39	0.68	0.00080	0.95
49	0.25	0.000090	0.34	0.00040	0.69	81	0.96
0.000050	0.25	91	0.34	41	0.70	82	0.96

Slope i	$f(i)$	Slope i	$f(i)$	Slope i	$f(i)$	Slope i	$f(i)$
0.00083	0.97	0.0034	1.70	0.0075	2.11	0.0137	2.27
84	0.97	35	1.71	76	2.11	0.0140	2.28
85	0.98	36	1.73	77	2.11	0.0142	2.28
86	0.98	37	1.75	78	2.12	0.0145	2.28
87	0.99	38	1.76	79	2.12	0.0147	2.28
88	0.99	39	1.78	0.0080	2.13	0.0150	2.28
89	1.00	0.0040	1.79	81	2.13	0.0152	2.28
0.00090	1.00	41	1.80	82	2.14	0.0155	2.28
91	1.01	42	1.82	83	2.14	0.0157	2.28
92	1.01	43	1.83	84	2.15	0.0160	2.28
93	1.02	44	1.84	85	2.16	0.017	2.28
94	1.02	45	1.85	86	2.16	0.018	2.28
95	1.03	46	1.87	87	2.16	0.019	2.28
96	1.03	47	1.88	88	2.17	0.020	2.28
97	1.04	48	1.89	89	2.17	0.030	2.28
98	1.04	49	1.90	0.0090	2.18	0.040	2.28
99	1.05	0.0050	1.91	91	2.18	0.050	2.28
0.0010	1.05	51	1.92	92	2.19	0.060	2.28
11	1.09	52	1.93	93	2.19	0.070	2.28
12	1.13	53	1.94	94	2.19	0.080	2.28
13	1.16	54	1.95	95	2.20	0.090	2.28
14	1.21	55	1.96	96	2.20	0.100	2.28
15	1.25	56	1.97	97	2.20		
16	1.28	57	1.98	98	2.21		
17	1.31	58	1.99	99	2.21		
18	1.34	59	2.00				
19	1.37	0.0060	2.01	0.0100	2.21		
0.0020	1.40	61	2.02				
21	1.43	62	2.02	0.0102	2.22		
22	1.45	63	2.03	0.0105	2.23		
23	1.48	64	2.04	0.0107	2.23		
24	1.50	65	2.04	0.0110	2.24		
25	1.52	66	2.05	0.0112	2.24		
26	1.55	67	2.06	0.0115	2.25		
27	1.57	68	2.07	0.0117	2.25		
28	1.59	69	2.07	0.0120	2.26		
29	1.61	0.0070	2.08	0.0122	2.27		
				0.0125	2.27		
0.0030	1.63	71	2.08	0.0127	2.27		
31	1.65	72	2.09	0.0130	2.27		
32	1.66	73	2.09	0.0132	2.27		
33	1.68	74	2.10	0.0135	2.27		

Values of the Function of Depth $f(t)$ for the Matakiewicz Formula

t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$
0.01	0.041	0.41	0.558	0.81	0.898	1.21	1.19	1.61	1.45	2.02	1.70
0.02	0.067	0.42	0.567	0.82	0.905	1.22	1.19	1.62	1.46	2.04	1.71
0.03	0.089	0.43	0.576	0.83	0.914	1.23	1.20	1.63	1.46	2.06	1.72
0.04	0.109	0.44	0.586	0.84	0.921	1.24	1.21	1.64	1.47	2.08	1.73
0.05	0.128	0.45	0.595	0.85	0.928	1.25	1.21	1.65	1.48	2.10	1.75
0.06	0.145	0.46	0.604	0.86	0.936	1.26	1.22	1.66	1.48	2.12	1.76
0.07	0.162	0.47	0.614	0.87	0.944	1.27	1.23	1.67	1.49	2.14	1.77
0.08	0.177	0.48	0.623	0.88	0.951	1.28	1.23	1.68	1.49	2.16	1.78
0.09	0.193	0.49	0.632	0.89	0.958	1.29	1.24	1.69	1.50	2.18	1.79
0.10	0.208	0.50	0.641	0.90	0.967	1.30	1.25	1.70	1.51	2.20	1.80
0.11	0.222	0.51	0.649	0.91	0.974	1.31	1.25	1.71	1.51	2.22	1.82
0.12	0.236	0.52	0.659	0.92	0.981	1.32	1.26	1.72	1.52	2.24	1.83
0.13	0.249	0.53	0.668	0.93	0.989	1.33	1.27	1.73	1.53	2.26	1.84
0.14	0.263	0.54	0.676	0.94	0.996	1.34	1.27	1.74	1.53	2.28	1.85
0.15	0.275	0.55	0.685	0.95	1.00	1.35	1.28	1.75	1.54	2.30	1.86
0.16	0.288	0.56	0.694	0.96	1.01	1.36	1.29	1.76	1.54	2.32	1.87
0.17	0.301	0.57	0.702	0.97	1.02	1.37	1.30	1.77	1.55	2.34	1.88
0.18	0.313	0.58	0.711	0.98	1.02	1.38	1.30	1.78	1.56	2.36	1.90
0.19	0.325	0.59	0.720	0.99	1.03	1.39	1.31	1.79	1.56	2.38	1.91
0.20	0.337	0.60	0.728	1.00	1.04	1.40	1.31	1.80	1.57	2.40	1.92
0.21	0.349	0.61	0.736	1.01	1.05	1.41	1.32	1.81	1.57	2.42	1.93
0.22	0.360	0.62	0.745	1.02	1.05	1.42	1.33	1.82	1.58	2.44	1.94
0.23	0.372	0.63	0.753	1.03	1.06	1.43	1.33	1.83	1.59	2.46	1.95
0.24	0.383	0.64	0.761	1.04	1.07	1.44	1.34	1.84	1.59	2.48	1.96
0.25	0.394	0.65	0.770	1.05	1.07	1.45	1.35	1.85	1.60	2.50	1.97
0.26	0.405	0.66	0.778	1.06	1.08	1.46	1.35	1.86	1.60	2.52	1.98
0.27	0.416	0.67	0.786	1.07	1.09	1.47	1.36	1.87	1.61	2.54	2.00
0.28	0.427	0.68	0.794	1.08	1.10	1.48	1.37	1.88	1.62	2.56	2.01
0.29	0.438	0.69	0.803	1.09	1.10	1.49	1.37	1.89	1.62	2.58	2.02
0.30	0.448	0.70	0.811	1.10	1.11	1.50	1.38	1.90	1.63	2.60	2.03
0.31	0.459	0.71	0.819	1.11	1.12	1.51	1.39	1.91	1.63	2.62	2.04
0.32	0.469	0.72	0.827	1.12	1.12	1.52	1.39	1.92	1.64	2.64	2.05
0.33	0.479	0.73	0.834	1.13	1.13	1.53	1.40	1.93	1.64	2.66	2.06
0.34	0.489	0.74	0.842	1.14	1.14	1.54	1.41	1.94	1.65	2.68	2.07
0.35	0.499	0.75	0.850	1.15	1.14	1.55	1.41	1.95	1.66	2.70	2.08
0.36	0.509	0.76	0.858	1.16	1.15	1.56	1.42	1.96	1.66	2.72	2.09
0.37	0.520	0.77	0.866	1.17	1.16	1.57	1.42	1.97	1.67	2.74	2.10
0.38	0.529	0.78	0.874	1.18	1.17	1.58	1.43	1.98	1.68	2.76	2.12
0.39	0.538	0.79	0.882	1.19	1.17	1.59	1.44	1.99	1.68	2.78	2.13
0.40	0.548	0.80	0.890	1.20	1.18	1.60	1.44	2.00	1.69	2.80	2.14

t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$	t in m	$f(t)$
2.82	2.15	3.62	2.56	5.05	3.07	7.05	3.32	9.05	3.57	11.05	3.82
2.84	2.16	3.64	2.57	5.10	3.07	7.10	3.32	9.10	3.57	11.10	3.82
2.86	2.17	3.66	2.58	5.15	3.08	7.15	3.33	9.15	3.58	11.15	3.83
2.88	2.18	3.68	2.59	5.20	3.08	7.20	3.33	9.20	3.58	11.20	3.83
2.90	2.19	3.70	2.60	5.25	3.09	7.25	3.34	9.25	3.59	11.25	3.84
2.92	2.20	3.72	2.61	5.30	3.10	7.30	3.35	9.30	3.60	11.30	3.85
2.94	2.21	3.74	2.62	5.35	3.10	7.35	3.35	9.35	3.60	11.35	3.85
2.96	2.22	3.76	2.63	5.40	3.11	7.40	3.36	9.40	3.61	11.40	3.86
2.98	2.23	3.78	2.64	5.45	3.12	7.45	3.37	9.45	3.62	11.45	3.87
3.00	2.24	3.80	2.65	5.50	3.12	7.50	3.37	9.50	3.62	11.50	3.87
3.02	2.25	3.82	2.66	5.55	3.13	7.55	3.38	9.55	3.63	11.55	3.88
3.04	2.26	3.84	2.67	5.60	3.13	7.60	3.38	9.60	3.63	11.60	3.88
3.06	2.27	3.86	2.68	5.65	3.14	7.65	3.39	9.65	3.64	11.65	3.89
3.08	2.28	3.88	2.68	5.70	3.15	7.70	3.40	9.70	3.65	11.70	3.90
3.10	2.29	3.90	2.69	5.75	3.15	7.75	3.40	9.75	3.65	11.75	3.90
3.12	2.30	3.92	2.70	5.80	3.16	7,80	3.41	9.80	3.66	11.80	3.91
3.14	2.32	3.94	2.71	5.85	3.17	7.85	3.42	9.85	3.67	11.85	3.91
3.16	2.33	3.96	2.72	5.90	3.17	7,90	3.42	9.90	3.67	11.90	3.92
3.18	2.34	3.98	2.73	5.95	3.18	7.95	3.43	9.95	3.68	11.95	3.93
3.20	2.35	4.00	2.75	6.00	3.18	8.00	3.43	10.00	3.69	12.00	3.94
3.22	2.36	4.05	2.77	6.05	3.19	8.05	3.44	10.05	3.69		
3.24	2.37	4.10	2.78	6.10	3.20	8.10	3.45	10.10	3.70		
3.26	2.38	4.15	2.80	6.15	3.20	8.15	3.45	10.15	3.70		
3.28	2.39	4.20	2.82	6.20	3.21	8.20	3.46	10.20	3.71		
3.30	2.40	4.25	2.84	6.25	3.22	8.25	3.47	10.25	3.72		
3.32	2.41	4.30	2.86	6.30	3.22	8.30	3.47	10.30	3.72		
3.34	2.42	4.35	2.87	6.35	3.23	8.35	3.48	10.35	3.73		
3.36	2.43	4.40	2.89	6.40	3.23	8.40	3.48	10.40	3.73		
3.38	2.44	4.45	2.90	6.45	3.24	8.45	3.49	10.45	3.74		
3.40	2.45	4.50	2.92	6.50	3.25	8.50	3.50	10.50	3.75		
3.42	2.46	4.55	2.94	6.55	3.25	8.55	3.50	10.55	3.75		
3.44	2.47	4.60	2.95	6.60	3.26	8.60	3.51	10.60	3.76		
3.46	2.48	4.65	2.97	6.65	3.27	8.65	3.52	10.65	3.77		
3.48	2.49	4.70	2.98	6.70	3.27	8.70	3.52	10.70	3.77		
3.50	2.50	4.75	3.00	6.75	3.28	8.75	3.53	10.75	3.78		
3.52	2.51	4.80	3.01	6.80	3.28	8.80	3.53	10.80	3.78		
3.54	2.52	4.85	3.03	6.85	3.29	8.85	3.54	10.85	3.79		
3.56	2.53	4.90	3.04	6.90	3.30	8.90	3.55	10.90	3.80		
3.58	2.54	4.95	3.05	6.95	3.30	8.95	3.55	10.95	3.80		
3.60	2.55	5.00	3.06	7.00	3.31	9.00	3.56	11.00	3.81		

Tables 26 and 27, showing the values $f(i)$ and $f(t)$ for the following formula, were elaborated by Matakiewicz to facilitate computation:

$$v = f(t) \times f(i)$$

The product of the values of the functions read on these tables represents the magnitude of a mean velocity.

Moreover, Matakiewicz presented Table 28, by which it is possible to determine the mean velocity if the depth and slope are measured.

Sokolovskii-Pokrovskii formula:

$$v = 17\, i^{0.4}\, t^{0.5}$$

where:

i — river slope expressed by a decimal fraction,

t — mean depth in m.

Average Velocities

i ‰	0.02	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
0.10	0.03	0.05	0.07	0.09	0.10	0.12	0.13	0.13	0.14	0.15	0.16	0.17
0.20	0.06	0.09	0.12	0.15	0.17	0.19	0.20	0.22	0.23	0.25	0.26	0.27
0.30	0.07	0.11	0.16	0.20	0.22	0.25	0.27	0.29	0.31	0.33	0.35	0.36
0.40	0.09	0.14	0.20	0.24	0.27	0.31	0.33	0.36	0.38	0.40	0.42	0.44
0.50	0.11	0.16	0.23	0.28	0.32	0.36	0.39	0.42	0.45	0.47	0.49	0.52
0.60	0.12	0.19	0.26	0.32	0.36	0.41	0.44	0.47	0.51	0.53	0.56	0.59
0.70	0.13	0.21	0.29	0.36	0.41	0.45	0.49	0.53	0.56	0.60	0.62	0.65
0.80	0.15	0.23	0.32	0.39	0.44	0.50	0.54	0.58	0.62	0.65	0.69	0.72
0.90	0.16	0.25	0.35	0.43	0.48	0.54	0.59	0.63	0.67	0.71	0.74	0.78
1.00	0.17	0.27	0.37	0.46	0.52	0.58	0.63	0.68	0.72	0.76	0.80	0.84
1.20	0.19	0.30	0.42	0.52	0.59	0.66	0.72	0.77	0.82	0.87	0.91	0.95
1.40	0.22	0.34	0.47	0.58	0.66	0.74	0.80	0.85	0.91	0.97	1.01	1.06
1.60	0.24	0.37	0.52	0.64	0.72	0.81	0.88	0.94	1.00	1.06	1.11	1.16
1.80	0.26	0.40	0.57	0.69	0.78	0.88	0.96	1.02	1.09	1.15	1.21	1.26
2.00	0.28	0.43	0.61	0.75	0.84	0.95	1.03	1.10	1.17	1.24	1.30	1.36
2.50	0.33	0.50	0.71	0.87	0.99	1.11	1.20	1.28	1.37	1.45	1.52	1.59
3.00	0.37	0.57	0.81	0.99	1.12	1.26	1.37	1.46	1.56	1.65	1.73	1.81
3.50	0.41	0.64	0.90	1.10	1.25	1.40	1.52	1.62	1.74	1.84	1.92	2.01
4.00	0.45	0.70	0.99	1.21	1.37	1.54	1.68	1.79	1.91	2.02	2.12	2.21
5.00	0.51	0.78	1.10	1.35	1.53	1.72	1.87	1.99	2.13	2.25	2.36	2.47
6.00	0.53	0.81	1.15	1.40	1.59	1.79	1.95	2.08	2.21	2.34	2.45	2.56
7.00	0.55	0.84	1.19	1.46	1.65	1.85	2.02	2.15	2.30	2.43	2.55	2.66
8.00	0.57	0.88	1.24	1.51	1.72	1.92	2.10	2.23	2.39	2.52	2.64	2.77
9.00	0.59	0.91	1.28	1.57	1.78	1.99	2.17	2.31	2.47	2.62	2.74	2.87
10.00	0.61	0.94	1.33	1.62	1.84	2.07	2.25	2.40	2.56	2.71	2.84	2.97

This formula was derived from measurements taken in 500 discharge section lines of various rivers with slopes lower than 0.1, and with channel distinctly shaped and not overgrown by vegetation.

Hermanek formulas (1905):

$$v = 30.7\, t \sqrt{i} = K_1 \sqrt{ti} \quad \text{for } t \leqslant 1.5 \text{ m},$$

$$v = 34\, t^{0.75} \sqrt{i} = K_2 \sqrt{ti} \quad \text{for } 1.5 < t < 6.0 \text{ m},$$

$$v = (50.2 + 0.5\, t) \sqrt{ti} = K_3 \sqrt{ti} \quad \text{for } t > 6.0 \text{ m},$$

where:

t — mean depth in m,
i — river slope expressed as a decimal fraction,
K_1, K_2, K_3 — coefficients presented in Table 29.

Table 28

according to Matakiewicz

0.60	0.70	0.80	0.90	1.00	1.50	2.00	2.50	3.00	3.50	4.00	5.00	i ‰ / t in m
0.18	0.19	0.20	0.21	0.22	0.26	0.28	0.32	0.34	0.36	0.37	0.40	0.10
0.28	0.30	0.32	0.34	0.35	0.42	0.47	0.51	0.55	0.58	0.60	0.65	0.20
0.38	0.40	0.43	0.45	0.47	0.56	0.63	0.68	0.73	0.77	0.80	0.86	0.30
0.46	0.49	0.52	0.55	0.58	0.68	0.77	0.84	0.89	0.94	0.98	1.05	0.40
0.54	0.58	0.61	0.64	0.68	0.80	0.90	0.98	1.04	1.10	1.15	1.23	0.50
0.61	0.66	0.70	0.73	0.77	0.91	1.02	1.11	1.19	1.25	1.30	1.39	0.60
0.68	0.73	0.77	0.82	0.86	1.01	1.14	1.24	1.32	1.39	1.45	1.55	0.70
0.75	0.80	0.85	0.89	0.94	1.11	1.25	1.36	1.45	1.53	1.59	1.70	0.80
0.81	0.87	0.92	0.97	1.02	1.21	1.35	1.47	1.58	1.66	1.73	1.85	0.90
0.87	0.94	0.99	1.05	1.10	1.30	1.46	1.59	1.70	1.78	1.86	1.99	1.00
0.99	1.06	1.13	1.19	1.24	1.47	1.65	1.80	1.92	2.02	2.11	2.26	1.20
1.10	1.18	1.26	1.32	1.39	1.64	1.84	2.00	2.14	2.26	2.35	2.52	1.40
1.21	1.30	1.38	1.45	1.52	1.81	2.02	2.20	2.36	2.48	2.59	2.77	1.60
1.32	1.41	1.50	1.58	1.66	1.96	2.20	2.39	2.56	2.69	2.81	3.01	1.80
1.42	1.52	1.61	1.70	1.78	2.11	2.37	2.58	2.76	2.90	3.03	3.24	2.00
1.66	1.78	1.88	1.98	2.08	2.47	2.77	3.01	3.22	3.39	3.54	3.78	2.50
1.89	2.02	2.14	2.26	2.37	2.81	3.14	3.42	3.66	3.85	4.02	4.30	3.00
2.10	2.25	2.39	2.51	2.64	3.12	3.50	3.81	4.07	4.29	4.47	4.79	3.50
2.31	2.48	2.63	2.76	2.90	3.44	3.85	4.19	4.48	4.72	4.92	5.27	4.00
2.57	2.76	2.93	3.08	3.23	3.83	4.29	4.67	5.00	5.26	5.49	5.87	5.00
2.68	2.87	3.04	3.20	3.36	3.92	4.46	4.86	5.19	5.46	5.70	6.10	6.00
2.78	2.98	3.16	3.33	3.49	4.14	4.63	5.05	5.40	5.68	5.92	6.34	7.00
2.89	3.09	3.28	3.45	3.62	4.29	4.81	5.24	5.60	5.89	6.15	6.57	8.00
2.99	3.20	3.40	3.58	3.76	4.45	4.98	5.43	5.80	6.11	6.37	6.82	9.00
3.10	3.32	3.52	3.71	3.89	4.61	5.16	5.63	6.01	6.33	6.61	7.07	10.00

Table 29

Values of the Coefficients of Velocity for the Hermanek Formula

t in m	K_1	t in m	K_2	t in m	K_2	t in m	K_3
0.1	9.7	1.6	38.1	4.0	47.9	7.0	53.7
0.2	13.7	1.8	39.4	4.2	48.6	8.0	54.2
0.3	16.8	2.0	40.5	4.4	49.3	9.0	54.7
0.4	19.4	2.2	41.5	4.6	49.6	10.0	55.2
0.5	21.7	2.4	42.2	4.8	50.3		
0.6	23.8	2.6	43.2	5.0	51.0		
0.7	25.8	2.8	43.9	5.2	51.3		
0.8	27.3	3.0	44.9	5.4	51.7		
0.9	29.2	3.2	45.6	5.6	52.4		
1.0	30.7	3.4	46.2	5.8	52.7		
1.2	33.8	3.6	46.9	6.0	53.0		
1.4	36.2	3.8	47.6				

Velocities Calculated

t in m \ i ‰	0.025	0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7
0.1	0.03	0.04	0.06	0.08	0.10	0.11	0.12	0.13	0.14
0.2	0.05	0.07	0.10	0.14	0.17	0.19	0.21	0.23	0.25
0.3	0.07	0.10	0.14	0.19	0.23	0.26	0.29	0.31	0.34
0.4	0.09	0.13	0.17	0.24	0.28	0.32	0.36	0.39	0.42
0.5	0.11	0.15	0.20	0.28	0.34	0.39	0.43	0.46	0.50
0.6	0.13	0.17	0.24	0.32	0.39	0.44	0.49	0.54	0.57
0.7	0.14	0.19	0.27	0.37	0.44	0.50	0.56	0.60	0.65
0.8	0.16	0.21	0.29	0.40	0.49	0.56	0.62	0.67	0.72
0.9	0.17	0.24	0.32	0.44	0.53	0.61	0.67	0.73	0.79
1.0	0.19	0.26	0.35	0.48	0.58	0.66	0.73	0.80	0.85
1.25	0.22	0.30	0.42	0.57	0.69	0.79	0.85	0.95	1.02
1.50	0.25	0.35	0.48	0.66	0.79	0.91	1.00	1.09	1.17
1.75	0.29	0.39	0.54	0.74	0.89	1.02	1.13	1.23	1.32
2.0	0.32	0.44	0.60	0.82	0.99	1.13	1.25	1.36	1.46
2.5	0.40	0.53	0.72	0.97	1.15	1.30	1.43	1.55	1.66
3.0	0.44	0.59	0.80	1.07	1.28	1.45	1.59	1.72	1.84
3.5	0.48	0.65	0.87	1.17	1.40	1.58	1.74	1.88	2.01
4.0	0.52	0.70	0.94	1.27	1.51	1.71	1.88	2.03	2.17
4.5	0.65	0.75	1.01	1.33	1.62	1.83	2.01	2.18	2.33
5.0	0.59	0.80	1.07	1.44	1.72	1.95	2.14	2.32	2.47

Gröger formulas (1913):

$$v = 23.781\, t^{0.776}\, i^{0.458} \text{ for } 0.2 < t < 2.0 \text{ m},$$

$$v = 22.110\, t^{0.58}\, i^{0.43} \text{ for } t > 2.0 \text{ m}$$

These formulas were derived on the basis of material collected in 940 discharge section lines. This formula can be applied if $B \geqslant 10$ m, $t > 0.2$ m, $i \leqslant 0.005$.

The values of the velocity computed by the Gröger formula are shown in Table 30.

Christen formula:

$$v = 6.307 \sqrt[8]{t\, i} \sqrt[8]{0.5\, B}$$

where:

t — mean depth in m,
i — slope of a water surface expressed by a decimal fraction,
B — width of a cross section in m.

Table 30

according to Groeger

0.8	0.9	1.0	1.25	1.5	2.0	2.5	3.0	3.5	4.0	5.0
0.15	0.16	0.17	0.19	0.20	0.23	0.26	0.28	0.30	0.32	0.35
0.26	0.28	0.29	0.32	0.35	0.40	0.44	0.48	0.52	0.54	0.62
0.36	0.38	0.40	0.44	0.48	0.54	0.60	0.65	0.71	0.75	0.83
0.45	0.47	0.49	0.55	0.59	0.68	0.75	0.82	0.88	0.93	1.03
0.53	0.56	0.59	0.65	0.71	0.81	0.89	0.97	1.04	1.11	1.23
0.61	0.64	0.68	0.75	0.81	0.93	1.03	1.12	1.20	1.28	1.41
0.69	0.73	0.76	0.84	0.92	1.05	1.16	1.26	1.35	1.44	1.59
0.76	0.81	0.85	0.94	1.02	1.16	1.29	1.40	1.50	1.60	1.77
0.84	0.88	0.93	1.03	1.12	1.27	1.41	1.53	1.64	1.75	1.94
0.91	0.96	1.01	1.11	1.21	1.38	1.53	1.62	1.78	1.90	2.10
1.08	1.14	1.20	1.32	1.44	1.64	1.82	1.98	2.12	2.26	2.50
1.24	1.31	1.38	1.53	1.66	1.89	2.10	2.28	2.44	2.60	2.88
1.40	1.48	1.55	1.72	1.87	2.13	2.36	2.57	2.75	2.93	3.24
1.55	1.64	1.72	1.91	2.07	2.36	2.62	2.85	3.06	3.25	3.60
1.75	1.84	1.93	2.12	2.30	2.60	2.86	3.09	3.31	3.50	3.85
1.95	2.05	2.14	2.36	2.55	2.89	3.18	3.44	3.68	3.89	4.28
2.13	2.24	2.35	2.58	2.79	3.16	3.48	3.76	4.02	4.26	4.69
2.30	2.42	2.53	2.79	3.02	3.41	3.76	4.06	4.34	4.60	5.06
2.47	2.59	2.71	2.99	3.23	3.66	4.02	4.35	4.65	4.92	5.42
2.62	2.76	2.88	3.17	3.43	3.87	4.28	4.63	4.94	5.23	5.76

The mean depth t is assumed in the formulas presented above instead of the hydraulic radius R. Such an assumption is correct if the river width is equal to at least 30 times the mean depth. This condition is usually fulfilled on rivers, but for computing ditches and narrow streams a hydraulic radius should be used instead of a mean depth.

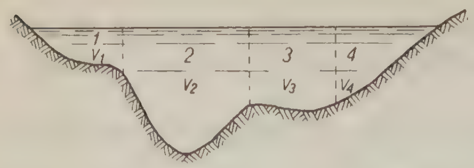


Fig. 66. A cross section of a river divided into four parts

If the cross section of a channel is not uniform, the channel should be divided into parts of an identical character, and mean velocis ties should be computed for each part (Fig. 66). Note that in computing the hydraulic radius of each part of a cros-section, only the length of a real contact of water with the channel sides and bottom should be considered as a wetted perimeter, while the contact of water on the border of the adjoining parts of a cross section is not taken into account.

Computing the Discharge

Computing discharge by the second group formula shown above consists in multiplying the determined velocity by the area of a cross section.

If the cross section of the channel was divided into parts, the discharge should be separately computed for each part by means of corresponding velocities and areas of cross sections. Partially summing the discharges, a total discharge is obtained in a cross section:

$$Q = Q_1 + Q_2 + Q_3 + Q_4$$

3. Empirical Methods of Computing Maximum Discharge

If no observations were made and no necessary measurements taken in the cross section of a river under study, the maximum discharges are mostly determined by taking corresponding values from other cross sections on the same river, by analogy to the adjoining rivers, or by means of empirical formulas.

Note that observations made over a short period — e. g. 2-3 years — are not sufficiently reliable for the characterization of streams and, therefore, for computing maximum discharges on such rivers the methods to be applied are the same as when no observation data are available.

Computing Discharge from Isoline Maps

Maximum discharges can be computed from isoline maps. The elaboration of such a map consists in plotting the values of a maximum unit runoff in the

cross section for which this value is known. Connecting the points with identical runoffs, lines are obtained of identical maximum runoffs (similar to the contour lines). The value of the maximum runoff in any cross section can be read by means of these lines.

Because no direct measurements and observation data are usually available for small rivers, isolines are drawn, primarily, on the basis of materials dealing with large rivers. Therefore a mechanical bisection of the small catchment basins with isolines, the points of which have been determined by values valid for large rivers, cannot yield satisfactory results for small catchment basins.

Small rivers are characterized by many specific features. Moreover, maximum unit runoff increases considerably with decrease in the catchment basin area. Therefore, it is incorrect to draw maps of isolines of maximum runoffs.

Isoline maps can be prepared only for determination of the approximate multiannual value of a mean runoff, called the standard runoff, which is considered to be independent of the area of the catchment basin. Since this is correct for catchment basins larger in area than 1000 sq km, the determination of a standard runoff for small catchment basins from the isoline maps may also result in considerable errors.

Computing Discharge by Terrain Coefficient

Efforts have been made by some investigators to correlate the magnitude of the maximum unit runoff q with the terrain coefficient A, identical for the same climatic area. The value of the terrain coefficient (climatic coefficient) A was shown on maps in the form of isolines.

The maximum unit runoff in such cases was mostly established by the following equation:

$$q = \frac{A}{F^n}$$

The maximum discharge was then computed from the following correlation:

$$Q = q\,F$$

It emerges from this correlation that increase in maximum discharge for the same area takes place along with increase in the catchment basin area; but that is not always so. A similar form can be given to the formula for computing an annual flow and not a maximum discharge.

To achieve greater accuracy in computations, the following formula for determining the value of the maximum discharge is recommended by Boldakov:

$$Q = AF^n B^m i^{0.25}$$

A value B (width of a catchment basin), characterizing the shape of a catch-

ment basin, the slope i, and taking into account the influence of the inclination, was introduced in the formula. The values of coefficients n and m are shown in Table 31

Table 31

Values of Coefficients n and m

Number	Cause of a rise	Direction of runoff	n	m
1	Snow thawing	Northwards	3 : 4	1 : 4
		Southwards	2 : 3	1 : 4
2	Rains		1 : 2	1 : 3

The width of a catchment basin B is obtained by dividing the area of a catchment basin by its length, but not by the length of a river.

The following equation for computing the terrain coefficient A may be derived from the formula for Q presented above:

$$A = \frac{Q}{F^n B^m i^{0.25}}$$

The corresponding values for drawing the isoline map of a climatic coefficient A for computing maximum discharge can be determined by the above correlation.

Transferring Discharges from Other Cross Sections of the Same River

If other cross sections, for which the values of maximum discharge directly or indirectly determined are known, are in the vicinity of a cross section under study, the values of such discharges may be transferred and applied to a cross section for which no observation data and measurements are available.

The following conditions should be observed in transferring discharges from elsewhere:

(a) the place the maximum discharges of which are known is located on the same river, below or above the cross section computed,

(b) there are no large tributaries (the catchment basins of which have areas equal to 10—15 percent of the entire area of a catchment basin belonging to the cross section computed) between the cross section under study and the cross section with a known maximum discharge.

Transferring discharges may be effected by the following methods:

(1) If the difference between the catchment basin area of the cross section computed, and the cross section with known value of maximum discharge is

insignificant (up to 3 percent), an identical discharge without any changes and conversions may be adopted in both such cross sections.

(2) If the difference is greater, discharges should be converted by one of the following formulas:

(a) maximum discharge in a cross section under study is computed by the known discharge in a cross section located above or below according to the following formula:

$$Q_x = \frac{Q_o F_x}{F_o}$$

where:

Q_x — the value of the maximum discharge sought,

Q_o — the known value of the maximum discharge in an adjoining cross section on the same river,

F_x, F_o — catchment basin areas in corresponding cross sections.

The above formula is based on an assumption, that discharges change proportionally to the area of a catchment basin; that assumption is not always correct and, therefore, the formula should be applied with great circumspection.

(b) more reliable results can be obtained when maximum discharges in the cross sections located above and below the cross section computed are known; the maximum discharge is then obtained from the following correlation:

$$Q_x = Q_1 + (Q_2 - Q_1)\frac{F_x - F_1}{F_2 - F_1}$$

where:

Q_x, F_x — maximum discharge and catchment basin area in a cross section computed,

Q_1, F_1 — maximum discharge and catchment basin area in an upper cross section,

Q_2, F_2 — maximum discharge and catchment basin area in a lower cross section.

Computing Discharges by Analogy

In some cases, in the vicinity of a river for which maximum discharges are to be found there may be situated another sufficiently investigated river for which the values of maximum discharges are known.

If catchment basins of these rivers are similar in their physio-geographical and geometrical characteristics, the unit runoff of the catchment basin investigated can serve for establishing the runoff in the catchment basin for which computations are required.

A river already investigated is in this case called "the analogue", and the

cross section for which multiannual observations, measurements and computations are available is called the fundamental cross section.

The method of full analogy consists in assuming values of maximum runoffs appearing in an analogue river as being reliable for the river under study. Transferring values of the runoff is effected without any changes or corrections. Such a procedure is admissible in the case of a very close similarity in the fundamental characteristic features of the two rivers, and particularly in the area of the catchment basin and the quantity of precipitation.

When not all the fundamental features of the two rivers are similar, computation by the analogy method is more difficult and less reliable. In this case, the value of the maximum runoff, taken from the fundamental cross section of the analogue river should be corrected to suit the existing conditions.

In spite of some discrepancies, which may arise in such a case, the results of computation by the method of analogy are usually closer to reality than those arrived at by empirical formulas.

The maximum runoff in a computed catchment basin can sometimes be determined also by the interpolation of runoffs of the two adjoining catchment basins. This method, which is similar to that of computing by isolines, yields, however, more accurate results because smaller terrain is here covered by interpolation than by interpolation from the isoline map prepared for large areas. This method is suitable rather for determining values for many years of the mean runoff in a catchment basin.

Empirical Formulas Based on the Characteristics of a Catchment Basin

Formula of the Former Ministry of Railroads

To compute the openings of small bridges and culverts for railroad purposes, the maximum unit runoffs presented in Table 32 have been recommended for use by the former Polish Ministry of Railroads.

The runoffs were correlated with the length of a catchment basin and its slope as well as with the terrain configuration.

It the valley of the stream is short (up to 3 km) and has steep slopes, the magnitude of the runoff should be increased 25 percent.

For a catchment basin covered with shrubs, or even having no vegetal cover at all, but located on highly permeable grounds, the runoff can be reduced, but by not more than 25 percent.

For forests, gravelly grounds and stony or sandy deserts, the runoff adopted may be reduced by 50 percent.

Formula of the Former Ministry of Public Works

For computing culverts and small bridges, the following empirical formula was recommended by the former Polish Ministry of Public Works:

Table 32

Maximum Runoffs for Small Catchment Basins in cu m/sec per sq km

Length of catchment basin in km	Terrain configuration of a catchment basin		
	mountain terrain	undulating terrain	flat terrain
	$i > 20^0/_{00}$	$20 - 5^0/_{00}$	$i < 5^0/_{00}$
1	8.0	6.4	4.0
2	7.0	5.6	3.5
3	6.0	4.8	3.0
4	5.0	4.0	2.5
6	4.0	3.2	2.0
10	3.0	2.4	1.5
14	2.0	1.6	1.0
18	1.0	0.8	0.5

$$Q = Pab \text{ cu m/sec}$$

where:

P — catchment basin area in sq km,

a — coefficient depending on the length of a catchment basin and the character of the terrain (Table 33),

b — coefficient depending on the degree of afforestation of a catchment basin (Table 34).

Table 33

Coefficients α

Length of catchment basin in km	Terrain configuration		
	mountain terrain	hilly terrain	flat terrain
1	7.0	5.6	3.5
3	5.8	4.6	2.9
5	4.8	3.8	2.4
7	4.0	3.2	2.0
10	3.0	2.4	1.5
15	2.0	1.6	1.0
20	1.4	1.1	0.7
25	1.0	0.8	0.5

The following formulas were derived by Iszkowski for computing the volume of particular types of discharges:

(1) discharge at mean annual water level:

$$Q_s = 0.03171\ C_s\, h\, F \quad \text{cu m/sec}$$

(2) discharge at the absolutely lowest water level:

$$Q_o = 0.2\, \gamma\ Q_s \qquad \text{cu m/sec}$$

(3) discharge at the average low level:

$$Q_1 = 0.4\, \gamma\ Q_s \qquad \text{cu m/sec}$$

(4) discharge at the normal level suitable for electric power purposes; in Poland this level lasts for 8-9 months in a year:

$$Q_2 = 0.7\, \gamma\ Q_s \qquad \text{cu m/sec}$$

(5) discharge at the highest level:

$$Q_3 = C_w\, m\, h\, F \ \text{cu m/sec}$$

(6) discharge at the disastrous level:

$$Q_4 = 1.3\, Q_3 \ \text{cu m/sec}$$

The following denotations have been used in the above formulas according to the category of catchment basin (Table 35):

Table 34

Coefficients b

Degree of afforestation of a catchment basin	b
0	1.0
0.25	0.9
0.50	0.8
0.75	0.7
1	0.6

C_s — coefficient of water runoff (Table 36),

C_w — coefficient depending on the size of the catchment basin, type of soil, vegetal cover and terrain configuration (Table 36),

m — coefficient depending on the size of the catchment basin (Table 37),

F — catchment basin area in sq km,

γ — coefficient depending on the size of the catchment basin, soil permeability and type of vegetal cover (Table 38),

h — height of annual layer of precipitation in m.

In these formulas for catchment basin area F smaller than 100 sq km in a flat terrain and smaller than 3000 sq km in a foothill and mountain terrain, Iszkowski recommends, if the real quantity of precipitation is lower[4], the adoption

[4]) The Iszkowski method has been shown here after the Czech hydrologists on the basis, primarily, of the book "Hydrology" by Novotny.

Table 35

Table for Determining Category of Catchment Basin according to Iszkowski

Catchment basin area in sq km	Plowed soil; highly permeable ground with average vegetal cover		Ground average permeable soil with average vegetal cover in lowland, foothill and mountain catchment basin		Impermeable ground with average vegetal cover in a mountain catchment basin or with steep hills			Completely impermeable ground in a mountain catchment basin or with steep hills; very wet or frozen ground
	small quantity of ground water	large quantity of ground water	flat or slightly undulating terrain	strongly undulating terrain	river basin lying not much above sea level	river basin lying higher above sea level	small river basin with steep gradients	
0–50	I	II	II	III	III	III	IV	IV
50–150	I	II	II	III	III	III	III–IV	IV
150–300	I	II	II	II–III	III	III	III–IV	IV
300–1000	I	II	II	II–III	III	III		
1000–4000	I	I–II	II	II	III	III		
4000–5000	I	I	II	II	III	III		
5000–12000	I	I	II	II	III	II–III		
above 12000	I	I	II	II	III	II		

Table 36

Coefficients C_s and C_w for the Iszkowski Formulas

No.	Terrain configuration	C_s	C_w for particular categories of catchment basin			
			I	II	III	IV
1	Marshes and plains	0.20	0.017	0.030	—	—
2	Lowland, plateaux	0.25	0.025	0.040	—	—
3	Lowland interspersed with hills	0.30	0.030	0.055	—	—
4	Gentle hills	0.35	0.035	0.070	0.125	—
5	Steep hills	0.40	0.040	0.082	0.155	0.40
6	Mountain prominences and knolls up to 100 m	0.45	0.045	0.100	0.190	0.45
7	Afforested mountains above 100 m high	0.50	0.050	0.120	0.225	0.50
8	High mountains such as Karkonosze, Beskidy	0.55	0.055	0.140	0.290	0.55
9	Highest mountains taking into account their gradient	0.60 0.65 0.70	0.060 0.070 0.080	0.160 0.185 0.210	0.360 0.460 0.600	0.60 0.70 0.80

Table 37

Values of the Coefficient m for the Iszkowski Formula

F in sq km	m	F in sq km	m	F in sq km	m	F in sq km	m
1	10.00	350	6.37	3500	3.350	80000	2.260
10	9.50	400	6.22	4000	3.250	90000	2.155
20	9.00	500	5.90	4500	3.200	100000	2.050
30	8.50	600	5.60	5000	3.125	110000	1.980
40	8.23	700	5.35	6000	3.103	120000	1.920
50	7.95	800	5.12	7000	3.082	130000	1.855
60	7.75	900	4.90	8000	3.060	140000	1.790
70	7.60	1000	4.70	9000	3.038	150000	1.725
80	7.50	1200	4.515	10000	3.017	160000	1.650
90	7.43	1400	4.320	20000	2.909	170000	1.575
100	7.40	1600	4.145	30000	2.801	180000	1.500
150	7.10	1800	3.960	40000	2.693	190000	1.425
200	6.87	2000	3.775	50000	2.575	200000	1.350
250	6.70	2500	3.613	60000	2.470	225000	1.170
300	6.55	3000	3.450	70000	2.365	250000	1.000

Table 38

Values of the Coefficient γ for the Iszkowski Formulas

No.	Catchment basin characteristics	Numerical values of the coefficient γ for areas of catchment basin in sq km			
		200 to 20,000	up to 200	20,000 to 50,000	50,000 to 100,000
	Catchment basins where impermeable grounds prevail		Coefficient diminishes 25%	Coefficient increases 15%	Coefficient increases 50%
1	On a plain with lakes and ponds	1.5			
2	On a plain without lakes and ponds	1.0			
3	In slightly undulating terrain	0.8			
4	In a foothill terrain and lower mountains	0.6–0.5			
5	In high mountains	0.3			
6	On bare mountain slopes	0.3–0.0			
	Catchment basins where permeable grounds prevail				
7	Average permeable grounds, normal vegetal cover	1.0			
8	Permeable grounds, rich vegetation	0.8			
9	Highly permeable grounds, poor vegetation	0.4			

Note: In maritime climates with precipitation uniformly distributed throughout the year, the value of the coefficient γ is increased 50 percent.

of the height of the annual layer of precipitation h amounting to minimum 1 m (1,000 mm).

Pareński Formula (1925)

Pareński divides all rivers into two groups, *A* and *B*, giving separate formulas for each group.

In group *A*, he includes rivers flowing from the mountains such as: the Vistula, the Dniester and their Carpathian tributaries. The rivers of this group are divided into seven categories depending on the elevation of their catchment basins above sea level.

Group *B* comprises other rivers, divided into categories depending on the configuration of their basins:

V *B* — rivers with hilly catchment basins as, for instance: the northern tributaries of the Dniester, the sources of the Bug, as far as the last Volhynian ridges, and the upper reaches of the rivers flowing from the Świętokrzyskie Mountains.

VI *B* — rivers unnamed above.

Values of Coefficient *m* as depending on Precipitation,

River category	Catchment basin configuration	Area of catchment basin in sq km							
		1			10000			20000	
		Permeability of soil of catchment basin							
		low	high	average	low	high	average	low	high
I A	Mountains more than 1500 m above sea level	22.2	19.8	21.0	21.78	19.42	20.6	21.4	19.0
II A	Mountains between 1500 m and 1000 m above sea level	19.8	17.5	18.6	19.42	17.17	18.25	19.0	16.84
III A	Mountains between 1000 m and 500 m above sea level	17.5	15.4	16.4	17.17	15.06	16.09	16.84	14.77
IV A	Submontane terrains up to 500 m above sea level	15.4	13.4	14.3	15.06	13.15	14.03	14.77	12.90
V A	Hills	13.4	11.7	12.5	13.15	11.53	12.27	12.90	11.32
VI A	Plains	11.7	10.5	11.0	11.53	10.31	10.8	11.32	10.12
VII A	Marshy plains	10.5	9.5	10.0	10.31	9.33	9.82	10.12	9.16
V B	Hills	4.25	2.75	3.5	4.19	2.71	3.45	4.12	2.68
VI B	Plains	2.75	1.5	2.0	2.71	1.49	1.98	2.68	1.48

Remarks: Coefficient should be increased one-half of the difference of the values between cate-

Formula for group A:

$$Q = mP^{3/5}$$

where $m = x^{4/3} + 10$ (x depends on the size of catchment basin)

Formula for group B:

$$Q = mP^{2/3}$$

where m $= x^{4/3} + 1.0$

Denotations:

Q — discharge volume in cu m/sec,

P — catchment basin area in sq km,

m — coefficient depending on precipitation, configuration and permeability of ground (selected from Table 39).

Configuration and Permeability of Terrain

Area of catchment basin in sq km												
	50000			100000			200000			500000		
Permeability of soil of catchment basin												
average	low	high	average	low	high	average	low	high	average	low	high	average
20.2	20.08	17.92	19.0	17.95	16.05	17.0	13.72	12.28	13.0	1.0	1.0	1.0
17.9	17.92	15.85	16.84	16.05	14.21	15.1	12.28	10.90	11.56	1.0	1.0	1.0
15.78	15.85	13.91	14.86	14.21	12.48	13.32	10.90	9.61	10.24	1.0	1.0	1.0
13.77	13.91	12.16	12.97	12.48	10.92	11.64	9.61	8.44	8.98	1.0	1.0	1.0
12.04	12.16	10.67	11.35	10.92	9.60	10.2	8.44	7.45	7.9	1.0	1.0	1.0
10.6	10.67	9.55	10.0	9.60	8.6	9.0	7.45	6.70	7.0	1.0	1.0	1.0
9.64	9.55	8.65	9.10	8.60	7.80	8.20	6.70	6.10	6.40	1.0	1.0	1.0
3.4	3.93	2.57	3.25	3.6	2.4	3.0	2.95	2.05	2.5	1.0	1.0	1.0
1.96	2.57	1.45	1.9	2.4	1.4	1.8	2.05	1.30	1.6	1.0	1.0	1.0

gories in montane catchment basins of a surface of 150 sq km and in lowland ones 250 sq km

The formulas set out above are applied in computing maximum discharges in all catchment basins.

Elaborating formulas of this type is difficult, and they do not yield satisfactory results since they cannot take into account the specific conditions prevailing in individual catchment basins.

Regional Formulas

More accurate and easier to derive are formulas adapted for computing maximum discharges on one river only, or in some strictly defined area (regional formulas). Some of these have the following forms.

Rybczyński established a formula for computing maximum discharges of the River Vistula:

$$Q = 21\sqrt{A} \text{ cu m/sec}$$

where: A — catchment basin area in sq km.

Another formula for computing maximum discharges of the Carpathian tributaries of the Vistula river was prepared by Matakiewicz in 1934:

$$Q = 10\, D^{0.6932} \text{ cu m/sec}$$

where: D — catchment basin area in sq km.

He also derived an approximate formula for computing maximum discharges of the River Vistula itself:

$$Q = 10.440 \sqrt{1 - \left(\frac{D - 193.014}{216.267}\right)^2} \text{ cu m/sec}$$

Empirical Formulas Based on the Intensity of Rains

There are also many formulas for computing maximum discharges with unknown probability of occurrence by means of the high intensity precipitation observed. Formulas of this type are derived for computing discharges in small catchment basins.

In computing maximum discharges by such formulas it is necessary on every occasion to obtain from appropriate authorities the values of maximum precipitation established on the basis of observations, and valid for a given area.

Specht Formula

The Specht formula is the best known and mostly commonly used among the formulas of this type. It was derived on the basis of observations of torrential rains in Bavaria, but it may also be applied to compute maximum discharges in Poland.

The Specht formula has the following general form:

$$q = N\left(0.2 + \frac{0.8}{\sqrt[4]{t+1}}\right)\frac{10}{36}\sqrt[12]{\frac{1}{F}}$$

where:

q — highest runoff in cu m/sec per sq km,

N — the most intensive precipitation observed in mm/hr, corresponding to the time of the afflux of a wave T or to the duration of rain t in the catchment basin having an area of F sq km,

F — catchment basin area in sq km,

t — duration (in hours) of the most intensive rain N or time of the afflux T

(concentration) of the tidal wave for the catchment basin having an area of F sq km and length L km ($t = \sqrt{cL} = T$),

c — coefficient of a value between 10 and 20, depending on the conditions of the water runoff; $c = 10$ for very good conditions and $c = 20$ for very poor conditions.

Table 40 presents the values of precipitation N_G and N_H in mm/hr, arranged in correlation with t.

Table 40

Magnitude of Precipitation in mm/hr

$t = T$	N_G	N_H	$t = T$	N_G	N_H
1	83	79	12	12	8
2	62	42	13	11.5	8
3	42	28.5	14	11	7.5
4	29	20	15	10.5	7.4
5	15.5	15.5	16	10	7
6	15	12.5	17	9.5	7
7	14.5	9	18	9.5	7
8	14	9	24	9	5.5
9	13.5	8.5	36	8	4
10	13	8.5	48	7	3
11	12.5	8	72	5.5	2

In this table:

N_G — the magnitude of precipitation for mountain vicinities with an annual precipitation betwen 800 and 2,000 mm,

N_H — the magnitude of precipitation for hilly areas with an annual precipitation between 500 and 800 mm.

With appropriate denotations of multipliers, the Spetch formula can be expressed in the following form:

$$Q = 0.278\, N\, k\, \alpha\, F$$

where:

Q — maximum discharge in cu m/sec,

N — height of a layer of precipitation in mm,

k — ratio of intensity of the mean precipitation to the maximum precipitation in a given area: $k = \frac{J_o}{J_{max}} = \sqrt[12]{\frac{1}{F}}$,

α — coefficient of runoff which is a function of the duration of rain t in hours: $\alpha = 0.2 + \frac{0.8}{\sqrt[4]{1 + t}}$,

F — area of catchment basin in sq km,

0.278 — commuter serving to convert the intensity of precipitation — expressed in millimeters per hour — into the volume of precipitation expressed in cu m/sec per sq km, i. e.:

$$\frac{1,000,000}{1,000 \times 3,600} = \frac{10}{30} = 0.278$$

Computing discharges by the Specht formula, we can use the tables worked out by the author of the formula. He divided the catchment basins of streams into two types:

(1) mountain basins with a depth of annual precipitation between 800 and 2,000 mm,

(2) hilly basins with precipitation between 500 and 800 mm.

According to this division, he elaborated a table of correlation between the intensity of precipitation and its most probable duration for various sizes of catchment basins (Table 41).

Starting from this table of maximum possible precipitation, Specht also prepared a table of corresponding runoffs using coefficient α, depending on the duration of rain t in hours (Table 42).

The absolute maximum can be computed from the table if we know the critical period of duration of rain T. Specht assumes the critical period of duration of rain from the beginning of disastrous precipitations to the afflux of the peak wave (water crest) according to the already given approximate formula established on the basis of the study:

$$T = \sqrt{cL}$$

Specht's formula yields satisfactory results for the length of a catchment basin area

$$20 \text{ km} < L < 100 \text{ km}$$

In catchment basins shorter than 20 km, the absolute maximum discharge is caused by short violent downpours during which the water runoff occurs at a considerable velocity, making it difficult to determine T.

For lengths of catchment basins exceeding 100 km, the main stream may have several tributaries, the influence of which should be taken into account separately in computations.

For approximate computations, Specht recommends the adoption of:

$T = \frac{L}{3}$ for steep slopes and higher velocities, and

$T = \frac{L}{2}$ at small velocities and lesser slopes.

The difficulty in determining accurately the duration of the most intensive rain constitutes the main drawback to Specht's formula, as well as to other formulas of this type. In comparison with other similar formulas, however, Specht's yields the best results and is easy to use in practice.

Table 41

Maximum Precipitation for sq km in cu m/sec depending on Surface Area and Configuration of Catchment Basin

Duration of the most intensive rain t in hrs	Mountain terrain									Hilly terrain								
	Catchment basin area in sq km									Catchment basin area in sq km								
	1	10	25	50	100	500	1000	5000	10000	1	10	25	50	100	500	1000	5000	10000
	Maximum precipitation in cu m/sec per sq km									Maximum precipitation in cu m/sec per sq km								
1	23.0	19.0	18.0	17.0	16.0	—	—	—	—	22.0	18.0	17.0	16.0	15.0	—	—	—	—
2	17.0	14.0	13.0	13.0	12.0	10.0	—	—	—	12.0	9.7	9.0	8.3	8.0	7.0	—	—	—
3	12.0	9.7	9.0	8.3	8.1	7.0	6.7	—	—	8.0	6.7	6.1	6.0	5.6	4.7	4.4	—	—
4	8.0	6.7	6.1	5.8	5.6	4.7	4.4	3.9	—	5.6	4.7	4.2	3.9	3.6	3.3	3.1	2.8	—
5	4.3	3.6	3.3	3.1	3.0	2.6	2.4	2.2	2.0	4.3	3.6	3.3	3.1	3.0	2.6	2.4	2.3	2.0
6	4.2	3.5	3.2	3.0	2.9	2.5	2.4	2.1	2.0	3.5	3.0	2.7	2.5	2.4	2.1	2.0	1.7	1.6
7	4.0	3.3	3.1	2.9	2.7	2.4	2.2	2.0	1.9	2.6	2.1	1.9	1.8	1.7	1.5	1.4	1.3	1.2
8	3.9	3.2	3.0	2.8	2.6	2.3	2.2	1.9	1.8	2.4	2.1	1.9	1.8	1.7	1.5	1.4	1.3	1.2
9	3.7	3.1	2.8	2.7	2.5	2.2	2.1	1.9	1.7	2.3	2.0	1.8	1.7	1.6	1.4	1.3	1.3	1.1
10	3.6	3.0	2.7	2.6	2.4	2.1	2.0	1.8	1.7	2.3	1.9	1.8	1.7	1.6	1.4	1.3	1.2	1.1
11	3.4	2.9	2.6	2.5	2.3	2.1	1.9	1.7	1.6	2.3	1.9	1.7	1.6	1.5	1.4	1.3	1.2	1.1
12	3.3	2.8	2.5	2.4	2.2	2.0	1.9	1.7	1.5	2.2	1.8	1.7	1.6	1.5	1.3	1.2	1.1	1.0
13	3.2	2.6	2.4	2.3	2.2	1.9	1.8	1.6	1.5	2.2	1.8	1.6	1.6	1.5	1.3	1.2	1.1	1.0
14	3.0	2.5	2.3	2.2	2.1	1.8	1.7	1.5	1.4	2.1	1.7	1.6	1.5	1.4	1.3	1.2	1.1	1.0
15	2.9	2.4	2.2	2.1	2.0	1.7	1.6	1.4	1.4	2.1	1.7	1.6	1.5	1.4	1.2	1.1	1.0	1.0
16	2.8	2.3	2.1	2.0	1.9	1.7	1.5	1.4	1.3	2.0	1.7	1.5	1.4	1.3	1.2	1.1	1.0	1.0
17	—	—	2.0	1.9	1.8	1.6	1.4	1.3	1.2	—	—	1.5	1.4	1.3	1.2	1.1	1.0	0.9
18	—	—	—	1.9	1.8	1.6	1.4	1.3	1.2	—	—	—	1.4	1.3	1.1	1.0	0.9	0.9
24	—	—	—	—	1.8	1.5	1.4	1.3	1.2	—	—	—	—	1.0	0.9	0.8	0.8	0.7
36	—	—	—	—	—	1.4	1.3	1.2	1.1	—	—	—	—	—	0.7	0.7	0.6	0.6
48	—	—	—	—	—	1.2	1.1	1.0	0.9	—	—	—	—	—	0.53	0.47	0.44	0.42
72	—	—	—	—	—	0.9	0.86	0.8	0.72	—	—	—	—	—	0.30	0.29	0.27	0.25

Table 42

Maximum Runoff per sq km in cu m/sec

Duration of the most intensive rain t in hr	Coefficient of flow α	Mountain terrain: Catchment basin area in sq km									Hilly terrain: Catchment basin area in sq km								
		1	10	25	50	100	500	1000	5000	10000	1	10	25	50	100	500	1000	5000	10000
		Maximum runoff in cu m/sec per sq km									Maximum runoff in cu m/sec per sq km								
1	0.90	21.0	17.0	16.0	15.0	14.0	—	—	—	—	20.0	16.0	15.0	14.0	13.0	—	—	—	—
2	0.80	14.0	11.0	10.0	10.0	9.6	8.0	—	—	—	9.6	7.8	7.2	6.6	6.4	5.6	—	—	—
3	0.77	9.2	7.5	6.9	6.4	6.2	5.4	5.2	—	—	6.2	5.2	4.7	4.6	4.3	3.6	3.4	—	—
4	0.74	5.9	5.0	4.5	4.3	4.1	3.5	3.3	2.9	—	4.1	3.5	3.1	2.9	2.7	2.4	2.3	2.1	—
5	0.71	3.0	2.6	2.3	2.2	2.1	1.8	1.7	1.6	1.4	3.0	2.6	2.3	2.2	2.1	1.8	1.7	1.6	1.4
6	0.69	2.9	2.4	2.2	2.1	2.0	1.7	1.6	1.4	1.4	2.4	2.1	1.9	1.7	1.6	1.4	1.4	1.2	1.1
7	0.68	2.7	2.3	2.1	2.0	1.8	1.6	1.5	1.4	1.3	1.8	1.4	1.3	1.2	1.2	1.0	1.0	0.9	0.8
8	0.66	2.6	2.1	2.0	1.8	1.7	1.5	1.4	1.3	1.2	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.9	0.8
9	0.65	2.4	2.0	1.8	1.7	1.6	1.4	1.3	1.2	1.1	1.5	1.3	1.2	1.1	1.0	0.9	0.8	0.8	0.7
10	0.64	2.3	1.9	1.7	1.6	1.5	1.4	1.3	1.2	1.1	1.5	1.2	1.2	1.1	1.0	0.9	0.8	0.8	0.7
11	0.63	2.1	1.8	1.6	1.6	1.4	1.3	1.2	1.1	1.0	1.4	1.2	1.1	1.0	0.9	0.9	0.8	0.8	0.7
12	0.62	2.0	1.7	1.6	1.5	1.4	1.2	1.2	1.1	0.9	1.4	1.1	1.1	1.0	0.9	0.8	0.7	0.7	0.6
13	0.61	2.0	1.6	1.5	1.4	1.3	1.2	1.1	1.0	0.9	1.3	1.1	1.0	1.0	0.9	0.8	0.7	0.7	0.6
14	0.61	1.8	1.5	1.4	1.3	1.2	1.1	1.0	0.9	0.8	1.2	1.0	0.9	0.9	0.8	0.8	0.7	0.7	0.6
15	0.60	1.7	1.4	1.3	1.2	1.2	1.0	1.0	0.8	0.8	1.2	1.0	0.9	0.9	0.8	0.7	0.7	0.6	0.6
16	0.59	—	1.3	1.2	1.2	1.1	1.0	0.9	0.8	0.8	—	1.0	0.9	0.8	0.8	0.7	0.6	0.6	0.6
17	0.59	—	—	1.2	1.1	1.0	0.94	0.83	0.8	0.7	—	—	0.9	0.8	0.8	0.7	0.6	0.6	0.5
18	0.58		—		1.1	1.0	0.93	0.81	0.75	0.7	—	—	—	0.8	0.75	0.64	0.58	0.52	0.5
24	0.56	—	—	—	—	1.0	0.82	0.78	0.73	0.67	—	—	—	—	0.56	0.50	0.45	0.42	0.39
36	0.52	—	—	—	—	—	0.73	0.68	0.62	0.57	—	—	—	—	—	0.36	0.36	0.33	0.31
48	0.50	—	—	—	—	—	0.60	0.55	0.50	0.45	—	—	—	—	—	0.25	0.23	0.22	0.21
72	0.47	—	—	—	—	—	0.42	0.40	0.38	0.34	—	—	—	—	—	0.14	0.13	0.12	0.12

CHAPTER V

COMPUTING MAXIMUM DISCHARGES WITH DEFINED PROBABILITY OF OCCURRENCE

The empirical formulas so far presented for computing maximum discharges with an unknown probability of occurrence have serious disadvantages; some principal factors are often omitted in them, or taken into account partially and not always in a proper manner.

However, supplementing these formulas or deriving others based on similar assumptions, is not advisable; these formulas should not be used at present since computations of openings are necessary for discharges with a defined probability of occurrence.

It should be realized that the highest discharge computed according to formulas of this type is accidental; it can appear in a given cross section of a river, say, once in a 1,000 years, in another — once in a 100 years, in yet a third — once in 25 years, etc. For these reasons, certain openings or spans computed for an accidental maximum discharge may turn out to be too large, others — too small. Building too large an opening causes an increase in construction expenses which cannot be justified by sound reasons, while a structure with insufficient dimensions may easily be destroyed during water rises; in the latter case, there will be losses involved in building a new structure, as well as inconvenience to traffic.

Computing openings for the highest random discharges determined can cause a situation in which, on the same road, some structures will easily be able to stand considerable floods occurring, say, once in 200 years, while others can be destroyed by much lower rises appearing, for instance, once in 25 years.

The necessity to maintain continual traffic involves ensuring that the discharge capacity of structures serving to give passage to water discharge is identical on roads and railway lines of equivalent importance to transport. For this reason, structures on a given road should be computed for the maximum discharges appearing during a certain assumed period — i.e., for discharges with identical probability of occurrence. The length of this period on roads or railway lines of equivalent importance is established in correlation with the importance of such routes.

1. Determining Probability of the Maximum Discharges in Correlation with the Importance of Structures

The magnitude of a period in which maximum discharges appear to be valid for the computations of bridge spans and culvert openings, has been correlated

with various factors, such as building costs of a structure and the losses involved in its destruction and reconstruction, the period during which the material of a structure is worn out. For instance, on hard surface roads computation of wooden bridges has been recommended for 25-year water, and on the same roads steel bridges — for 100-year water.

The most rational way is to correlate the period of occurrence of varying frequency rises with the importance of a road. On this assumption, structures located on the same road should, irrespective of the material of which they are built, be computed for discharge with identical probability of occurrence. A traffic stoppage on a road or railway line must not be allowed to result from wooden bridges being destroyed by water — an eventuality to be envisaged if wooden bridges are built with spans smaller than the openings in structures made of more durable materials.

A traffic stoppage is always equally costly, irrespective of whether the bridge suffering destruction is built of wood, reinforced concrete or other material. Users will not be satisfied by being told that it was caused by the destruction of a bridge built of wood and not of steel.

High costs are, of course, involved in building structures which can give passage to very high rises. Therefore, for economic reasons, it is more rational to accept the fact that there is a risk of some bridges being destroyed and to stop traffic on less important transport routes if too high a rise appears.

Thus it is seen that correlating the probability with the material of which bridges are built is incorrect and that in computing spans, the degree of importance of the road should have prime consideration.

On the basis of these premises, the present author prepared Table 42, by means of which embankments, bridges and culverts on railways and roads of equivalent importance are computed for the same probability.

The following should constitute guiding principles for establishing probability:

(a) structures of identical kind located on the same road should be computed for discharges with identical probability, because thus identical durability of structures and their resistance to water discharge are achieved;

(b) bridges and culverts situated on the same road should be computed in principle for different probability of rises, since culverts resist higher rises than bridges and, therefore, culvert openings can be comparatively smaller;

(c) embankments can be computed for lower probability of discharges than bridges and culverts;

(d) road embankments should be computed for higher probability than that accepted for railway embankments because road embankments

are usually broader than railway embankments and have a hard surface giving them greater resistance to water pressure and filtration.

Note that the probability of rises as assumed in Table 43 is entirely acceptable for the present conditions. It may in future prove necessary to compute structures located on very important routes for rises with lower probability.

Computing hydrotechnical structures for 1,000-year water has been started in some countries. Later on, water appearing once in 100,000 years was assumed in more important cases, and even „maximum-maximorum" water of a discharge, the magnitude of which cannot be exceeded in nature. It can be assumed that this is a discharge with a probability of occurrence once in a 1,000,000 years.

This giving consideration to discharges of a very rare occurrence was prompted by the fear that the possible destruction — however improbable — of some very important hydrotechnical structures might cause a great number of casualties and enormous economic losses.

2. The Theory of Probability

In computing bridges and culverts, the maximum discharges are established by means of the theory of probability adopting the highest discharge which may appear in a certain defined period. For instance, bridges on national highways should be computed for a maximum discharge having an average occurrence once in 100 years (Table 43).

Applying the theory of probability, the maximum discharges valid for the computation of structure openings can be established by one of the following three methods:

Table 43

Assumed Frequencies of Discharges in Correlation with the Importance of Structures

No.	Railway lines or roads	Embankments		Bridges		Culverts	
		Probability					
		1 : n	%	1 : n	%	1 : n	%
1	Broad-gaged railroads	1 : 600	0.167	1 : 300	0.333	1 : 100	1
2	Narrow-gaged railroads	1 : 100	1	1 : 50	2	1 : 25	4
3	Temporary railroads and detours	1 : 50	2	1 : 25	4	1 : 10	10
4	National highways and streets constituting extensions of such	1 : 100	1	1 : 100	1	1 : 50	2
5	County and district roads and civic streets unnamed in point 4	1 : 50	2	1 : 50	2	1 : 25	4
6	Communal roads	1 : 25	4	1 : 25	4	1 : 10	10
7	Temporary roads and detours	1 : 10	10	1 : 10	10	1 : 10	10

(1) by systematic multiannual hydrological observations in the cross section of the bridge or the adjoining cross sections,
(2) by short series of observations and various additional hydrological data,
(3) by empirical formulas.

It is advisable to determine the maximum discharges necessary for computing openings of bridges and culverts by the first and second method; empirical formulas should be applied only to the preliminary computations.

In practice, however, maximum discharges are computed by all the above mentioned methods.

Maximum discharges can appear as a result of:

(a) disastrous precipitation in warm seasons of the year following violent downpours or prolonged rains called summer rains,
(b) snow thawing on a frozen ground,
(c) snow thawing on a frozen ground and simultaneous rains.

Snow thawing in spring with simultaneous rains usually causes higher rises than snow thawing alone.

It may be assumed that maximum rises in streams appear as a result of summer or spring rains.

It was supposed at first that maximum discharges in a warm season of the year can be caused in the plains by violent downpours also. Recently, however, this supposition has been proved correct for very small areas only; in catchment basins of average size, maximum rises can also appear following prolonged rains.

Another supposition used to be that only in small catchment basins of area less than 60 sq km can summer rains cause rises higher than those produced by snow thawing. It has been found, however, that in every geographical area there is a limit magnitude of catchment basin, at which may be expected as a result of rains, the appearance of maximum discharges higher than those caused by snow thawing. The further southwards, the higher the increase in the magnitude of the catchment basin in which the maximum discharges may be caused by summer rains.

The magnitudes of the maximum discharges in a river depend on the coordination of a large number of simultaneously acting factors which may appear in various combinations.

The thickness of layer and degree of porosity of the snow, its distribution on the surface of a catchment basin, quantity of precipitation and temperature of air during the snow thawing, appearance of ground frosts in the night, the thickness of the frozen layer of the ground, ground water fed to a river, etc. — all these belong to the factors on which depends, for instance, the magnitude of a spring rise.

In some cases, those factors cause rises of an exceptional degree, while only an insignificant rise in the water level can be caused by the same factors if they occur in a different combination.

A change in any of these factors, influencing the formation of flow, depends strictly on the phenomena appearing in nature, while the magnitude of the maximum discharge formed by such factors is a purely random value.

This contingency may easily be proved by analizing the highest maximum discharges (e.g. spring discharges) year by year over a very long series of observations. Since in this case even an approximate order of appearance of maximum discharges cannot be proved on every river in particular years, it is obvious that these discharges are entirely independent one of the other. Note, that with an increase in the number of years covered by observations the mean value of maximum annual discharges for each river changes only very insignificantly.

The method of mathematical statistics based on the theory of probability serves for the investigation of these random values of maximum discharges (or other hydrological phenomena). This method may be used for the approximate determination of deviations of maximum annual discharges from mean values, as well as for finding the percentage of the probability of occurrence if the maximum discharge cannot be determined by the theory of probability, but the number of times the maximum discharge of a defined magnitude will probably be repeated during the period under study can be indicated with sufficient accuracy.

It was at first supposed that observation results from a period of at least 50 to 60 years are necessary to determine rises with a very low probability. Long periods of observation occur only on certain large rivers and in the majority of cases the period of observations, particularly on small rivers, is much shorter.

For this reason, detailed investigations were undertaken to establish the magnitude of errors committed in computations of probability on the basis of shorter periods of observation. It was found that a much shorter period of observation (10 to 15 years) appropriately selected is sufficient for determining rises with low probability.

The results of computations by statistical methods should always be checked to remove major errors, which are usually unavoidable with the schematization of phenomena necessary in this case.

Particular attention should be paid to the selection of initial data for statistical computations. In this connection the conditions of runoff for every individual year should be subjected to detailed analysis in order to explain the influence of such factors as, for instance, water elevation following ice jams, which can entirely distort the true picture of the course of the maximum rise.

An appropriate reduction in the ice gorge stage should be made in order to determine the stage valid for the computation of the maximum discharge without the rise caused by the ice gorge. For this purpose, a diagram should be prepared of the water stage fluctuations in the cross section under study, and in the nearest water gage cross section (in which there is no ice gorge) on the same river. The shape of the curve of water stage fluctuations in a cross

section under study during the period of occurrence of an ice gorge should be similar to that of water fluctuations in an adjoining cross section during the same period. The highest water level during the period of a rise, read on the curve thus drawn, will represent the stage appropriate for computing the maximum spring discharge.

The conditions of runoff can be completely changed by human activities in a catchment basin. Investigation should be made, therefore, as to whether such a change has taken place, since indiscriminately to take into account observations from periods with various conditions of runoff is incorrect.

Human activities aimed at river realignment, building dams and other structures causing a change in the river regimen, cannot be considered accidental. The statistical method can be used, therefore, for computing hydrological phenomena on rivers without structures provided that observation materials dealing with such rivers are free from the influence of factors deforming the phenomena.

Distributive Series

A certain quantity of numbers independent of each other but formed as a result of the action of homogeneous factors and characterizing some phenomenon is called the distributive series. The individual numbers of a series are arranged according to magnitude in decreasing or increasing order, to facilitate investigation.

For rivers, which as a rule have one major rise in a year, the quantity of numbers in a series is identical to the number of years during which observations are made. Such conditions usually exist in catchment basins where maximum rises are caused by snow thawing.

On rivers fed principally by summer rains, several major rises may appear during a year. In this connection, all the major rises which have occurred over a certain period are sometimes introduced to the series. Here, however, the computation acquires the character of a free choice because it is difficult to connect the probability of discharges with the length of the observation period.

Only one maximum rise in a year is usually taken into account, therefore, for computing the highest discharges caused by summer rains. The most appropriate, however, is to take into account the highest rises — for example, summer rises during a period studied. The number of such summer rises should be equal to the number of years during which observations are made.

In applying the method of mathematical statistics, only one annual maximum was taken into account in a distributive series for computing probability, irrespective of whether it was caused by snow thawing or summer rains.

This is a wrong approach, because mathematical statistics deals with the investigations of those series whose numbers have arisen following the ac-

tion of homogeneous factors and, therefore, discharges of different origin should not be interposed.

On this basis, in computations of probability of summer discharges, study is made in the Soviet Union of those factors which caused the occurrence of the maximum discharge observed, and the highest summer discharges of identical origin appearing in a year are introduced into the distributive series. For instance, the highest discharge for any year caused by monsoons is assumed irrespective of the fact that in some years higher discharges, though of different origin, were observed in summer.

Under Polish conditions, it is best to construct two separate series, one for the spring and one for the summer (including summer and fall) rises. In this event it should be stressed that as a result of taking into consideration two rises in a year, the probability of such as are determined in each series will be approximately doubled.

A number of auxiliary operations are usually necessary for computing probability of discharges. If, for instance, we have results of observation of the highest spring rises for a series of years, the corresponding discharges should be the first to be determined. These discharges are read on the curve drawn to represent the discharge.

Note that the accuracy of determination of reliable discharges by means of the theory of probability usually depends to a greater extent on the correct extrapolation of the discharge curve than on the method of computation, or even on the number of years during which observations are made.

Particular attention should be given to the preparation and extrapolation of the discharge rating curves.

There are several methods of extrapolating discharge rating curves — presented by Polyakov, Kravchenko and others. The most commonly used methods are those already discussed in previous chapters, and consisting in computing stream cross section and velocity by formulas of the Chézy and other types.

A curve of relation of the water gages — which shows the correlation between water stages in particular water gaging cross sections (Fig. 67) — serves for transferring highest water stages from one river cross section to another.

To establish the correlation, water stages occurring in one (usually upper) cross section are plotted on the vertical axis of coordinates, and corresponding water stages in another cross section — on the horizontal axis. Several points, generally scattered will arise at the bisection of perpendicular straight lines drawn in places where stages correspond one with another. A line (straight or curved) approximated on the basis of these points facilitates the establishment of a water stage on one water gage if the stage indicated by another water gage is known.

Elaborating a curve of relation of water gages, the water stages corresponding

one with another and shown by the two water gages are plotted on the diagram taking into consideration the time required for the water discharge to pass from one cross section to another. If, therefore, the selected cross sections are located at a distance passed by a wave in, say, one day, the water stage of the previous day will be the corresponding stage for a lower cross section.

Using the relation curve of water gages, we can draw a curve of the discharge for the bridge cross section *B*, if a curve of discharge exists in an adjoining cross section *A* (Fig. 68).

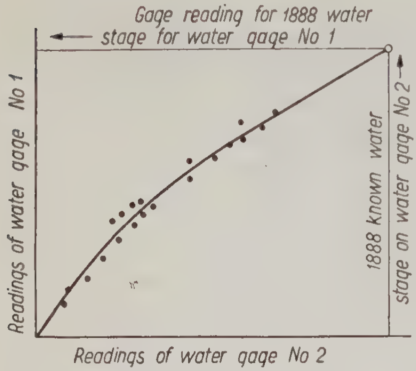


Fig. 67. Relation curve of water levels

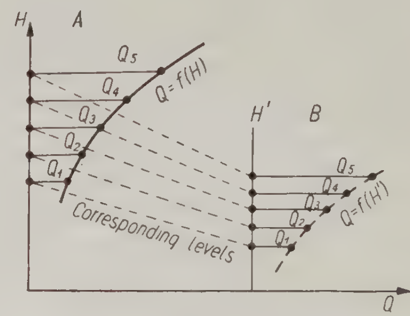


Fig. 68. Transferring the discharges from one curve to another

For this purpose, stages are drawn in the bridge cross section *B*, corresponding with the stages in the adjoining cross section *A* and, at points thus obtained, on the axis of coordinates the discharges corresponding one with another are plotted.

If the river sector between these cross sections has no tributaries and the increase in the area of the catchment basin is insignificant, the discharges in the two cross sections will be identical — $Q_1, Q_2 \ldots Q_5$. In the event of greater increases in the catchment basin area, discharges should be increased or decreased corresponding to the ratio of the areas of the two catchment basins.

Discharge Frequency Diagrams

After drawing the discharge rating curve, the discharges corresponding with the highest water stages in a year are read on it. These discharges — usually arranged in decreasing order — form a distributive series.

The full fluctuation amplitude of the highest annual discharges is divided into intervals (e.g. every 50 cu m/sec) and the number of discharges (frequence) for each interval is established in percentages or years (Table 44).

Plotting values of discharges in cu m/sec on the axis of ordinates of the diagram and the percentage probability of their occurrence on the axis of ab-

scissae, we obtain the diagram of frequency which is step-like in shape (Fig. 69a). As a result of connecting the centers of columns thus obtained by an undulating line, a curve of frequency is formed, also called a curve of duration.

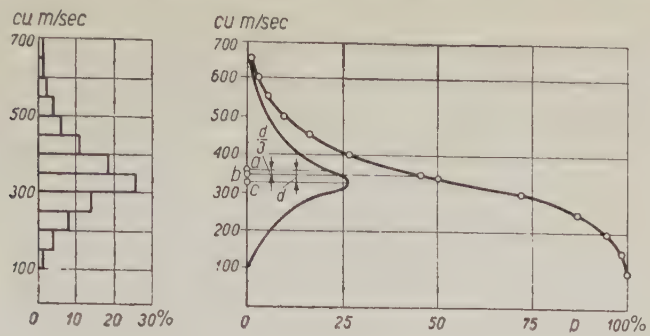


Fig. 69. The frequency curve and the cumulative curve

Steadily adding one to another the individual frequencies in years or as percentages (Table 44), we arrive at numbers indicating frequencies of discharges together with higher discharges.

Table 44

Data for Plotting Discharge Probability Curves

Discharges in cu m/sec	Probability of discharge		Probability of discharge together with higher discharges	
	in years	in percentages	in years	in percentages
700—650	1	1.3	1	1.3
649—600	1	1.3	2	2.6
599—550	2	2.6	4	5.2
549—500	3	4.0	7	9.2
499—450	5	6.7	12	15.9
449—400	8	10.7	20	26.6
399—350	14	18.7	34	45.3
349—300	20	26.7	54	72.0
299—250	11	14.7	65	86.7
249—200	6	8.0	71	94.7
199—150	3	4.0	74	98.7
149—100	1	1.3	75	100.0
	75	100		

Plotting in the diagram (Fig. 69b) the individual totals of percentage frequencies, and connecting the points thus obtained, we form the totalling or in-

tegral curve of frequency, also called a curve of the sum of the duration of discharge or a cumulative curve.

Three characteristic points are marked on the axis of coordinates of the frequency curve.

The first point *a* on this axis is called the central point and a line perpendicular to the axis and passing through this point is the central line. The central point corresponds with the arithmetic mean value of the distributive series.

The second point *b* expresses the value of a median. The line perpendicular to the axis of coordinates passing through this point bisects the area of the curve of frequency into equal parts. The median point corresponds with the central value of the distributive series (50 percent probability).

The third point *c* is called the modal point and a line drawn through this point and perpendicular to the axis is the modal line. The modal point corresponds to the largest number of observations in the distributive series.

It should be stressed that the curve of frequency in Figure 69 was drawn in a vertical position, although it is permissible in practice to show it in a horizontal position. The vertical position of the curve of frequency is aimed at a more accurate explanation of the method of obtaining the integral curve of frequency (cumulative curve).

The curves of frequency can be symmetric or asymmetric (skew), whereas the variation in the hydrological phenomena is usually characterized by the skew curves (Fig. 69).

The radius of skewness *d* representing the distance between the central and modal points (points *a* and *c* in Figure 69,) serves for determining the degree of skewness of such curves. The distance between the central and median points amounts to approximately $\frac{d}{3}$.

If the ordinates in the central, median and modal points are identical — i.e. if all these points coincide one with another on the axis of coordinates of the frequency curve, such a curve is symmetric, and an ordinate passing through such points is an axis of symmetry. The radius of skewness of such a curve is equal to zero.

Discharges of any probability of occurrence can be established by means of the integral curve of frequency.

Usually, the number of observations is in practice insufficient for drawing an accurate curve of frequency. The variation of some hydrological phenomena (e.g. discharge) cannot be fully characterized in a short time. In this connection it became necessary to select a theoretical curve of frequency which should depict with sufficient accuracy the character of variation of the hydrological phenomena and, at the same time, facilitate drawing the integral curve of frequency for a longer period. The extrapolation of a distributive series would thus be facilitated.

It has been shown by investigations that the Pearson type III theoretical curve of frequency, selected from among 14 curves of this type elaborated by Pearson, comes nearest to the real curve of frequency of discharges based on several observations.

The integral curve of the discharge frequency corresponding with the Pearson type III curve is skew. The lower part of this curve reaches 100 percent at a certain lowest value of discharge, while its upper part lies infinitely distant.

This does not conform with the real picture of the phenomenon, because the maximum discharge in a river cannot increase to infinity and must have a definite maximum extremal value.

The ends of the theoretical integral curve of frequency are steeply inclined, while the central part of the curve is mildly sloping. This proves that discharges with values approaching the arithmetic mean of the series occur much more frequently than do discharges departing from this value — i.e. grouping in the vicinity of the extremal values.

The equation of the Pearson type III curve has been accepted for valid in computing discharges because theoretical curves corresponding with this equation approximate very closely to the curves drawn by direct measurements, and the numerical parameters of this equation are fully justified.

The similarity between these curves enables us to suppose that extrapolation outside the limits of observation would yield satisfactory results.

Note that the Pearson equation has no theoretical justification. It constitutes a sort of empirical formula yielding a curve of a required shape.

Every effort should be made to obtain the greatest possible number of observations of extremal values of the maximum discharges because the extrapolation of the series will then be more reliable.

To conduct extrapolation — the results of which are less than accurate — the lowest and highest values of the real observations are essential, and only if major errors are found in them should they be disregarded.

The series of the maximum annual discharges are distinguished by the fact, that the values of discharges most often observed (corresponding with the maximum ordinate of the curve of frequency) are somewhat lower than the arithmetic mean of the value of the series. Skewness of this type is called positive skewness, by contrast with negative skewness, which appears when the most often observed numbers of the series are higher than the mean value. For instance, the series of maximum annual water stages in rivers almost always have negative skewness.

Statistical Parameters of the Series

The Pearson type III curve of frequency, applied by Foster to draw the curve of frequency of discharges, is expressed by the following equation:

$$y = y_0 e^{-\gamma x}\left(1 + \frac{x}{a}\right)^{-\gamma a},$$

where:

y_0 — highest (modal) ordinate,

e — base of natural logarithms,

γ — inversion of the radius of asymmetry $\gamma = \frac{1}{d}$,

a — distance between modal ordinate and the end of the curve,

y — (and x) — coordinates in a system whose axes bisect at the modal point.

The Pearson type III curve is fully determined for a given hydrological series by means of three constant parameters:

(a) arithmetic mean value of series Q_o — i.e. the mean of the highest discharge,

(b) coefficient of variation of series C_v,

(c) coefficient of skewness of series C_s.

Arithmetic Mean Value of Series

The arithmetic mean value of the series Q_0 is the principal and simplest characteristic of every series of random values. The mean arithmetic value of the series Q_0 is computed from the formula:

$$Q_0 = \frac{\sum Q}{n}$$

where:

$\sum Q$ — sum of all numbers of a series of maximum discharges,

n — the quantity of numbers of a series.

The mean arithmetic value Q_0 does not, however, sufficiently characterize a distributive series. Thus, for instance, if we decrease the highest numbers of a series by a value A and increase the lowest ones by the same value, the mean arithmetic value of a series will remain unchanged, although the character of such a series will be quite different.

Coefficient of Variation of the Series

The coefficient of variation C_v serves for characterizing the variation in the individual series. The higher the variation of the numbers of series, the higher the value of C_v.

The coefficient of variation of the maximum annual discharges is expressed by the following formula:

$$C_v = \frac{\sigma}{Q_0}$$

where σ — standard deviation of discharges from the mean arithmetic value Q_0.

The standard deviation is computed by means of the formula known from the theory of errors:

$$\sigma = \sqrt{\frac{\Sigma (Q_i - Q_o)^2}{n}}$$

The formula for computing the coefficient of variation is obtained by dividing the value by Q_o:

$$C_v = \sqrt{\frac{\Sigma (Q_i - Q_o)^2}{Q_o^2 n}} = \sqrt{\frac{\Sigma \left(\frac{Q_i}{Q_o} - 1\right)^2}{n}}$$

$$C_v = \sqrt{\frac{\Sigma (k-1)^2}{n}}$$

where $k = \frac{Q_i}{Q_o}$ is a coefficient expressing the ratio of every number (of discharge) of a series to the mean arithmetic value.

This formula serves for computing the coefficient of variation of the distributive series, consisting of the quantity of numbers exceeding 25. If the quantity of numbers in a series is lower, as often happens when studying maximum discharges, the coefficient of variation is computed from the formula:

$$C_v = \sqrt{\frac{\Sigma(k-1)^2}{n-1}}$$

The mean errors of the coefficient of variation determined by the formulas, given above and computed by Foster, are shown in Table 45.

Table 45

Values of Errors of the Coefficients C_v and C_s Depending on the Number of Observations

Number of years during which observations were made	Error of C_v in %	Error of C_s in %
5	5.0÷8.0	70÷83
10	2.0÷3.5	49÷51
20	1.2÷2.0	29÷32
100	0.3÷0.4	8÷11

In the event of a complete absence of observations of maximum discharges or of insufficient numbers of such the coefficients of variation for the spring

maxima can be computed approximately by the Polyakov, Sokolovski and Shevyelev formulas.

Thus, for instance, Polyakov established a correlation between the coefficient of variation C_v of the maximum discharges and the coefficient of variation C'_v of the mean annual discharges expressed by the following formula:

$$C_v = 1.97\,(C'_v)^{0.73}$$

A formula for computing C_v for large catchment basins is presented by Sokolowski:

$$C_v = a - 0.063\,log\,(F + 1)$$

where:

F — catchment basin area in sq km,

a — physio-geographical parameter, for the computation of which the formula $a = 0.73 - 0.29\,log\,M_0$ was derived by Shevyelov, where M_0 — mean modulus of the runoff.

The Shevyelov formula for computing C_v in small catchment basins has the following form:

$$C_v = a - 0.03\,log\,(F + 1)$$

Denotations identical with those in the Sokolovski formula are used in Shevyelov's formula.

The value of the coefficient of variation C_v for the series consisting of the maximum spring discharges can be computed by the approximate formulas presented above.

Coefficients of variation for the summer maxima have not thus far been investigated; such data as there are enable only the supposition that C_v for the maximum summer discharges is higher than C_v for the spring maxima, and fluctuates within fairly narrow limits.

It has been proved, that the value of the arithmetic mean of the distributive series Q_0 and the coefficient of variation C_v change hardly at all in the event of an increased number of observations. Therefore, it is assumed, that the computation of C_v may be made with sufficient accuracy when observations covering 10 to 15 years are available.

Coefficient of Skewness of a Series

The following formula serves for computing the coefficient of skewness:

$$C_s = \frac{\sum (k - 1)^3}{(n - 1)\,C_v^3}$$

The skewness of the curve of frequency is characterized by the coefficient of skewness C_s. For an accurate determination of the coefficient C_s by means of the formula presented above, a series is necessary which consists of a 100 and

even a greater quantity of numbers because such coefficient assumes its constant value very slowly.

The number of observations of maximum discharges is usually insufficient in practice to arrange such a long series; the coefficient C_s cannot, therefore, be computed by the formula already shown, but is determined as a rule in an indirect manner.

To determine the coefficient of skewness, a certain property of the Pearson type III curve was used consisting in the following correlation between the coefficients of variation and skewness:

$$\frac{C_s}{C_v} = \frac{2}{1 - k_{min}}$$

where:

$k_{min} = \frac{Q_{min}}{Q_0}$ — the lowest coefficient of modulus or the ratio of the lowest of the maximum discharges in a series to the mean discharge.

The following formula for computing the coefficient of skewness may easily be obtained from this correlation:

$$C_s = \frac{2 C_v}{1 - k_{min}}$$

When observations over a sufficiently long period are not available, Soviet 1948 standards recommend:

$C_s = 2\ C_v$ — for the spring maxima,

$C_s = 4\ C_v$ — for the summer maxima.

Professor Sokolovski concluded that in computing maximum discharges for afforested and marshy catchment basins, $C_s = 2\ C_v$ can be used; for lowland rivers — $C_s = 3\ C_v$; and for mountain rivers — $C_s = 4\ C_v$.

In practice, C_s is very often computed on the basis of the correlation $C_s = 2\ C_v$. Such a premise is not always correct for small rivers, although it is often close enough to the true values for large rivers.

It is impossible to determine, from an insignificant number of observations, accurate values of C_s for small catchment basins. For this reason, in computations of runoff from small catchment basins, methods of computing C_s should be the same as for large catchment basins.

Meteorological data can be used for approximate computations. Thus, for instance, rivers with a high coefficient of flow (mountain rivers) should have the value C_s for flow fluctuations which approaches the corresponding value C_s for precipitations.

3. Determining Probability of Discharge on the Basis of Observations

The probability of the occurrence of various hydrological phenomena may be accurately determined from a long period of observations.

Using Table 46 (elaborated by Foster) a theoretical integral curve of frequency is drawn, based on observations.

This curve may be checked by plotting on the graph the values of maximum discharges for the corresponding percentage probability.

The integral curve of probability can be drawn in a usual scale or, to increase the accuracy of the diagram, in the scale of probability.

Mathematical Statistics Method of Determining the Integral Probability Curve

The theoretical integral curve of probability based on the Pearson type III curve can be drawn after determining the values of the coefficients of variation C_v and skewness C_s.

The ordinates of this curve are computed from Table 46, prepared by Foster for values C_s contained within limits of 0 and 3.00. Later, the Foster table was extended and supplemented by Rybkin for values C_s between 0 and 2.0.

The ordinates of the total tables are given in the form of deviations f from the number one, at various values C_s and $C_v = 1$.

Since these deviations are proportional to C_v, therefore the numbers shown in Table 46 should be multiplied by the corresponding C_v to obtain the values of deviations with other values of C_v, being different from one.

As the linear interpolation in this case does not yield accurate values, diagrams were elaborated by the present author and should be applied for computing probability, which was not given by the Foster table (Figs. 70, 71, 72).

The following order should be observed in computing maximum discharge with defined frequency by the method of statistical mathematics called by hydrologists the Foster method:

(1) computing the mean artihmetic value of the series consisting of all the known observations of maximum discharges;

(2) establishing coefficients of modulus $k = \dfrac{Q}{Q_o}$ for every number of a series;

(3) determining the deviation of a coefficient of modulus from one $(k - 1)$ for each such coefficient;

(4) squaring the value $k - 1$ and summing up all numbers thus obtained;

(5) computing C_v from the formulas:

$$C_v = \sqrt{\frac{\sum (k-1)^2}{n}} \quad \text{or} \quad C_v = \sqrt{\frac{\sum (k-1)^2}{n-1}}$$

(6) establishing value C_s mostly by the formula:

$$C_s = \frac{2\,C_v}{1 - k_{min}}$$

(7) assuming from Table 46 appropriate values of the deviation f for

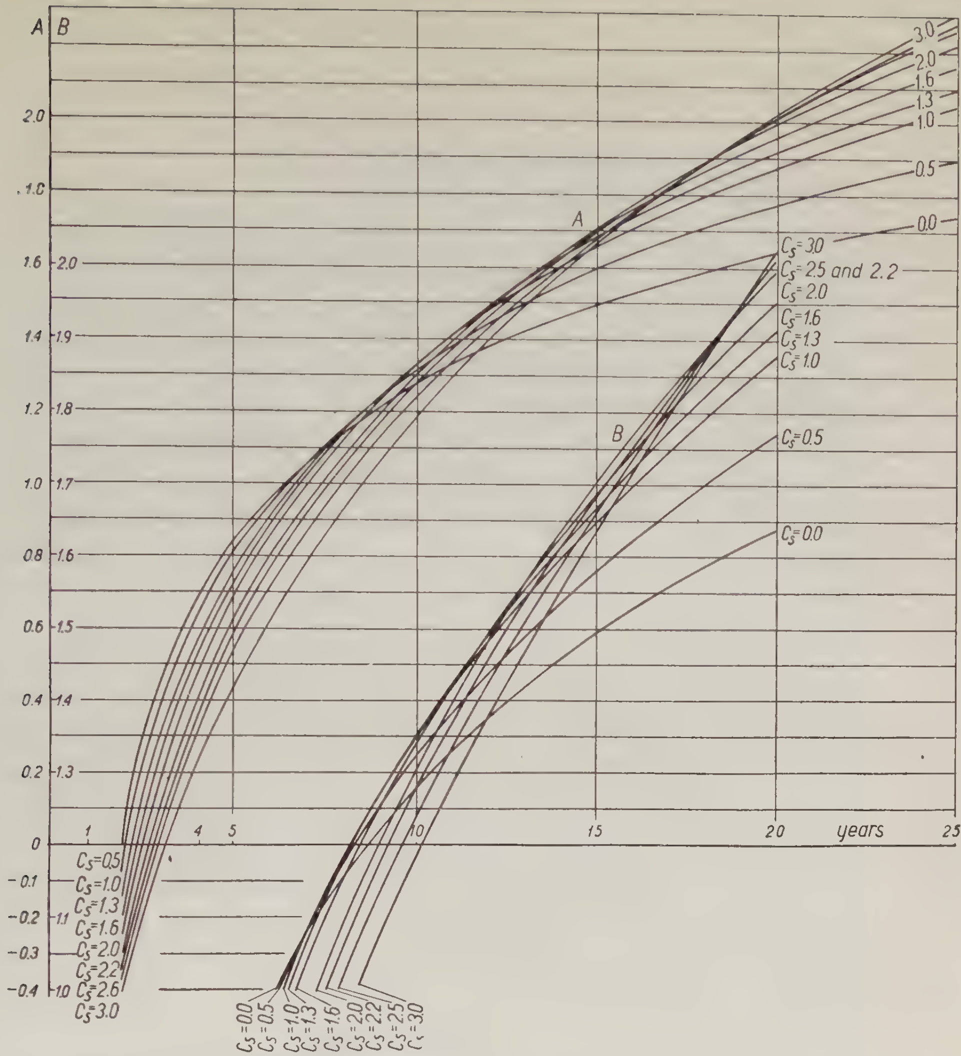


Fig. 70. Diagrams for computing probability of occurrence of discharge by the Foster method: A — for probability between 1 : 2 and 1 : 25 years, B — diagram in the increased scale for the central sector of curve A.

various probabilities and, subsequently, computing coefficient k from the formula:

$$k = f\,C_v + 1$$

(8) obtaining maximum discharges with various frequencies by multiplying coefficient k by the mean arithmetic value of a series:

$$Q = Q_o k = Q_o\,(f\,C_v + 1)$$

	Probability												
%	0.01	0.03	0.05	0.1	0	0.5	1	3	5	10	20	25	30
C_s \ 1:n	10000	3333	2000	1000	333	200	100	33	20	10	5	4	3.3
0.00	3.72	3.43	3.29	3.09	2.75	2.58	2.33	1.88	1.64	1.28	0.84	0.67	0.52
0.05	3.83	3.53	3.38	3.16	2.80	2.63	2.36	1.90	1.65	1.28	0.84	0.66	0.52
0.10	3.94	3.62	3.46	3.23	2.85	2.67	2.40	1.92	1.67	1.29	0.84	0.66	0.51
0.15	4.05	3.71	3.54	3.31	2.91	2.72	2.44	1.94	1.68	1.30	0.84	0.66	0.50
0.20	4.16	3.80	3.62	3.38	2.96	2.76	2.47	1.96	1.70	1.30	0.83	0.65	0.50
0.25	4.27	3.89	3.70	3.45	3.02	2.81	2.50	1.98	1.71	1.30	0.82	0.64	0.49
0.30	4.38	3.98	3.79	3.52	3.07	2.86	2.54	2.00	1.72	1.31	0.82	0.64	0.48
0.35	4.50	4.08	3.88	3.59	3.13	2.91	2.58	2.02	1.73	1.32	0.82	0.64	0.48
0.40	4.61	4.17	3.96	3.66	3.18	2.95	2.61	2.04	1.75	1.32	0.82	0.63	0.47
0.45	4.72	4.26	4.04	3.74	3.24	3.00	2.64	2.06	1.76	1.32	0.82	0.62	0.46
0.50	4.83	4.35	4.12	3.81	3.29	3.04	2.68	2.08	1.77	1.32	0.81	0.62	0.46
0.55	4.94	4.45	4.20	3.88	3.35	3.09	2.72	2.10	1.78	1.32	0.80	0.62	0.45
0.60	5.05	4.54	4.29	3.96	3.40	3.13	2.75	2.12	1.80	1.33	0.80	0.61	0.44
0.65	5.16	4.63	4.38	4.03	3.45	3.18	2.78	2.14	1.81	1.33	0.80	0.60	0.44
0.70	5.28	4.72	4.46	4.10	3.50	3.22	2.82	2.15	1.82	1.33	0.78	0.59	0.43
0.75	5.39	4.82	4.54	4.17	3.56	3.27	2.86	2.16	1.83	1.34	0.78	0.58	0.42
0.80	5.50	4.91	4.63	4.24	3.61	3.31	2.89	2.18	1.84	1.34	0.78	0.58	0.41
0.85	5.62	5.01	4.72	4.31	3.67	3.36	2.92	2.20	1.85	1.34	0.78	0.58	0.40
0.90	5.73	5.10	4.80	4.38	3.72	3.40	2.96	2.22	1.86	1.34	0.77	0.57	0.40
0.95	5.84	5.19	4.88	4.46	3.77	3.45	2.99	2.24	1.87	1.34	0.76	0.56	0.39
1.00	5.96	5.28	4.97	4.53	3.82	3.49	3.02	2.25	1.88	1.34	0.76	0.55	0.38
1.05	6.07	5.38	5.05	4.60	3.87	3.54	3.06	2.26	1.88	1.34	0.75	0.54	0.37
1.10	6.18	5.47	5.13	4.67	3.92	3.58	3.09	2.28	1.89	1.34	0.74	0.54	0.36
1.15	6.30	5.57	5.22	4.74	3.98	3.62	3.12	2.30	1.90	1.34	0.74	0.53	0.36
1.20	6.41	5.66	5.30	4.81	4.03	3.66	3.15	2.31	1.91	1.34	0.73	0.52	0.35
1.25	6.52	5.75	5.38	4.88	4.08	3.70	3.18	2.32	1.92	1.34	0.72	0.52	0.34
1.30	6.64	5.84	5.46	4.95	4.13	3.74	3.21	2.34	1.92	1.34	0.72	0.51	0.33
1.35	6.76	5.94	5.54	5.02	4.18	3.79	3.24	2.36	1.93	1.34	0.72	0.50	0.32
1.40	6.87	6.03	5.63	5.09	4.23	3.83	3.27	2.37	1.94	1.34	0.71	0.49	0.31
1.45	6.98	6.12	5.72	5.16	4.28	3.87	3.30	2.38	1.94	1.34	0.70	0.48	0.30

Table 46

of the Integral Curve of Probability at $C_v = 1.0$

	Probability												
%	40	50	60	70	75	80	90	95	97	99	99.5	99.7	99.9
C_s \ 1:n	2.5	2	0.4	0.3	0.25	0.20	0.1	0.05	0.03	0.01	0.005	0.003	0.001
0.00	0.25	0.00	−0.25	−0.52	−0.67	−0.84	−1.28	−1.64	−1.88	−2.33	−2.58	−2.75	−3.09
0.05	0.24	−0.01	−0.26	−0.52	−0.68	−0.84	−1.28	−1.67	−1.86	−2.29	−2.53	−2.69	−3.02
0.10	0.24	−0.02	−0.27	−0.53	−0.68	−0.85	−1.27	−1.61	−1.84	−2.25	−2.48	−2.64	−2.95
0.15	0.23	−0.02	−0.28	−0.54	−0.68	−0.85	−1.26	−1.60	−1.82	−2.22	−2.43	−2.58	−2.88
0.20	0.22	−0.03	−0.28	−0.55	−0.69	−0.85	−1.26	−1.58	−1.79	−2.18	−2.39	−2.53	−2.81
0.25	0.21	−0.04	−0.29	−0.56	−0.70	−0.85	−1.25	−1.56	−1.77	−2.14	−2.34	−2.47	−2.74
0.30	0.20	−0.05	−0.30	−0.56	−0.70	−0.85	−1.24	−1.55	−1.75	−2.10	−2.29	−2.42	−2.67
0.35	0.20	−0.06	−0.30	−0.56	−0.70	−0.85	−1.24	−1.53	−1.72	−2.06	−2.24	−2.36	−2.60
0.40	0.19	−0.07	−0.31	−0.57	−0.71	−0.85	−1.23	−1.52	−1.70	−2.03	−2.20	−2.31	−2.54
0.45	0.18	−0.08	−0.32	−0.58	−0.71	−0.85	−1.22	−1.51	−1.68	−2.00	−2.15	−2.26	−2.47
0.50	0.17	−0.08	−0.33	−0.58	−0.71	−0.85	−1.22	−1.49	−1.66	−1.96	−2.11	−2.21	−2.40
0.55	0.16	−0.09	−0.34	−0.58	−0.72	−0.85	−1.21	−1.47	−1.64	−1.92	−2.06	−2.15	−2.32
0.60	0.16	−0.10	−0.34	−0.59	−0.72	−0.85	−1.20	−1.45	−1.61	−1.88	−2.02	−2.10	−2.27
0.65	0.15	−0.11	−0.35	−0.60	−0.72	−0.85	−1.19	−1.44	−1.59	−1.84	−1.97	−2.05	−2.20
0.70	0.14	−0.12	−0.36	−0.60	−0.72	−0.85	−1.18	−1.42	−1.57	−1.81	−1.93	−2.00	−2.14
0.75	0.13	−0.12	−0.36	−0.60	−0.72	−0.86	−1.18	−1.40	−1.54	−1.78	−1.88	−1.95	−2.08
0.80	0.12	−0.13	−0.37	−0.60	−0.73	−0.86	−1.17	−1.38	−1.52	−1.74	−1.84	−1.90	−2.02
0.85	0.12	−0.14	−0.38	−0.60	−0.73	−0.86	−1.16	−1.36	−1.49	−1.70	−1.79	−1.85	−1.96
0.90	0.11	−0.15	−0.38	−0.61	−0.73	−0.85	−1.15	−1.35	−1.47	−1.66	−1.75	−1.80	−1.90
0.95	0.10	−0.16	−0.38	−0.62	−0.73	−0.85	−1.14	−1.34	−1.44	−1.62	−1.70	−1.75	−1.84
1.00	0.09	−0.16	−0.39	−0.62	−0.73	−0.85	−1.13	−1.32	−1.42	−1.59	−1.66	−1.71	−1.79
1.05	0.08	−0.17	−0.40	−0.62	−0.74	−0.85	−1.12	−1.30	−1.40	−1.56	−1.62	−1.66	−1.74
1.10	0.07	−0.18	−0.41	−0.62	−0.74	−0.85	−1.10	−1.28	−1.38	−1.52	−1.58	−1.62	−1.68
1.15	0.06	−0.18	−0.42	−0.62	−0.74	−0.84	−1.09	−1.26	−1.36	−1.48	−1.54	−1.57	−1.63
1.20	0.05	−0.19	−0.42	−0.63	−0.74	−0.84	−1.08	−1.24	−1.33	−1.45	−1.50	−1.53	−1.58
1.25	0.04	−0.20	−0.42	−0.63	−0.74	−0.84	−1.07	−1.22	−1.30	−1.42	−1.46	−1.49	−1.53
1.30	0.04	−0.21	−0.43	−0.63	−0.74	−0.84	−1.06	−1.20	−1.28	−1.38	−1.42	−1.45	−1.48
1.35	0.03	−0.22	−0.44	−0.64	−0.74	−0.84	−1.05	−1.18	−1.26	−1.35	−1.38	−1.41	−1.44
1.40	0.02	−0.22	−0.44	−0.64	−0.73	−0.83	−1.04	−1.17	−1.23	−1.32	−1.35	−1.37	−1.39
1.45	0.01	−0.23	−0.44	−0.64	−0.73	−0.82	−1.03	−1.15	−1.21	−1.29	−1.31	−1.33	−1.35

%	Probability												
%	0.01	0.03	0.05	0.1	0	0.5	1	3	5	10	20	25	30
C_S \ 1:n	10000	3333	2000	1000	333	200	100	33	20	10	5	4	3.3
1.50	7.09	6.21	5.80	5.23	4.33	3.91	3.33	2.39	1.95	1.33	0.70	0.47	0.30
1.55	7.20	6.30	5.88	5.30	4.38	3.95	3.36	2.40	1.96	1.33	0.69	0.46	0.29
1.60	7.31	6.39	5.96	5.37	4.42	3.99	3.39	2.42	1.96	1.33	0.68	0.46	0.28
1.65	7.42	6.48	6.04	5.44	4.47	4.03	3.42	2.43	1.96	1.32	0.67	0.45	0.27
1.70	7.54	6.57	6.12	5.50	4.52	4.07	3.44	2.44	1.97	1.32	0.66	0.44	0.26
1.75	7.65	6.66	6.20	5.57	4.57	4.11	3.47	2.45	1.98	1.32	0.65	0.43	0.25
1.80	7.76	6.75	6.28	5.64	4.62	4.15	3.50	2.46	1.98	1.32	0.64	0.42	0.24
1.85	7.87	6.84	6.36	5.70	4.67	4.19	3.52	2.48	1.98	1.32	0.64	0.41	0.23
1.90	7.98	6.93	6.44	5.77	4.71	4.23	3.55	2.49	1.99	1.31	0.63	0.40	0.22
1.95	8.10	7.02	6.52	5.84	4.76	4.27	3.58	2.50	2.00	1.30	0.62	0.40	0.21
2.00	8.21	7.11	6.60	5.91	4.81	4.30	3.60	2.51	2.00	1.30	0.61	0.39	0.20
2.05				5.99			3.63	2.52	2.00	1.30	0.60	0.39	0.20
2.10				6.06			3.65	2.53	2.00	1.29	0.60	0.38	0.19
2.15				6.13			3.68	2.54	2.01	1.28	0.59	0.38	0.18
2.20				6.20			3.70	2.55	2.01	1.28	0.58	0.37	0.17
2.25				6.27			3.72	2.56	2.01	1.27	0.57	0.36	0.16
2.30				6.34			3.75	2.56	2.01	1.27	0.56	0.35	0.15
2.35				6.40			3.77	2.56	2.01	1.26	0.55	0.34	0.14
2.40				6.47			3.79	2.57	2.01	1.25	0.54	0.33	0.13
2.45				6.54			3.81	2.58	2.01	1.25	0.54	0.32	0.13
2.50				6.60			3.83	2.58	2.01	1.24	0.53	0.32	0.12
2.55				6.67			3.85	2.58	2.01	1.23	0.52	0.31	0.11
2.60				6.73			3.87	2.59	2.01	1.23	0.51	0.30	0.10
2.65				6.80			3.89	2.59	2.01	1.22	0.50	0.29	0.09
2.70				6.86			3.91	2.60	2.01	1.21	0.49	0.28	0.08
2.75				6.92			3.93	2.61	2.02	1.21	0.48	0.27	0.07
2.80				6.99			3.95	2.61	2.02	1.20	0.47	0.27	0.06
2.85				7.05			3.97	2.62	2.02	1.20	0.46	0.26	0.05
2.90				7.12			3.99	2.62	2.02	1.19	0.45	0.26	0.04
2.95				7.18			4.00	2.62	2.02	1.18	0.44	0.25	0.04
3.00				7.25			4.02	2.62	2.02	1.18	0.42	0.25	0.03

	Probability												
%	40	50	60	70	75	80	90	95	97	99	99.5	99.7	99.9
C_s \ $1:n$	2.5	2	0.4	0.3	0.25	0.20	0.1	0.05	0.03	0.01	0.005	0.003	0.001
1.50	0.00	−0.24	−0.45	−0.64	−0.73	−0.82	−1.02	−1.13	−1.19	−1.26	−1.28	−1.30	−1.31
1.55	−0.01	−0.24	−0.46	−0.64	−0.73	−0.82	−1.00	−1.12	−1.16	−1.23	−1.25	−1.26	−1.28
1.60	−0.02	−0.25	−0.46	−0.64	−0.73	−0.81	−0.99	−1.10	−1.14	−1.20	−1.22	−1.23	−1.24
1.65	−0.02	−0.26	−0.46	−0.64	−0.72	−0.81	−0.98	−1.08	−1.12	−1.17	−1.18	−1.19	−1.20
1.70	−0.03	−0.27	−0.47	−0.64	−0.72	−0.81	−0.97	−1.06	−1.10	−1.14	−1.15	−1.16	−1.17
1.75	−0.04	−0.28	−0.48	−0.64	−0.72	−0.80	−0.96	−1.04	−1.08	−1.12	−1.12	−1.13	−1.14
1.80	−0.05	−0.28	−0.48	−0.64	−0.72	−0.80	−0.94	−1.02	−1.06	−1.09	−1.10	−1.10	−1.11
1.85	−0.06	−0.28	−0.48	−0.64	−0.72	−0.80	−0.93	−1.00	−1.04	−1.06	−1.07	−1.07	−1.08
1.90	−0.07	−0.29	−0.48	−0.64	−0.72	−0.79	−0.92	−0.98	−1.01	−1.04	−1.04	−1.05	−1.05
1.95	−0.08	−0.30	−0.48	−0.64	−0.72	−0.78	−0.91	−0.96	−0.99	−1.02	−1.02	−1.02	−1.02
2.00	−0.08	−0.31	−0.49	−0.64	−0.71	−0.78	−0.90	−0.95	−0.97	−0.99	−1.00	−1.00	−1.00
2.05	−0.09	−0.32	−0.49	−0.64	−0.71	−0.77	−0.89	−0.94	−0.95	−0.96			−0.97
2.10	−0.10	−0.32	−0.49	−0.64	−0.70	−0.76	−0.88	−0.93	−0.93	−0.94			−0.95
2.15	−0.10	−0.32	−0.49	−0.63	−0.70	−0.76	−0.86	−0.92	−0.92	−0.92			−0.93
2.20	−0.11	−0.33	−0.49	−0.63	−0.69	−0.75	−0.85	−0.90	−0.90	−0.90			−0.91
2.25	−0.12	−0.34	−0.49	−0.63	−0.68	−0.74	−0.83	−0.88	−0.88	−0.89			−0.89
2.30	−0.12	−0.34	−0.49	−0.62	−0.68	−0.73	−0.82	−0.86	−0.86	−0.87			−0.87
2.35	−0.13	−0.34	−0.50	−0.62	−0.67	−0.72	−0.81	−0.84	−0.84	−0.85			−0.85
2.40	−0.14	−0.35	−0.50	−0.62	−0.66	−0.71	−0.79	−0.82	−0.82	−0.83			−0.83
2.45	−0.14	−0.36	−0.50	−0.62	−0.66	−0.70	−0.78	−0.80	−0.80	−0.82			−0.82
2.50	−0.15	−0.36	−0.50	−0.61	−0.65	−0.70	−0.77	−0.79	−0.79	−0.80			−0.80
2.55	−0.16	−0.36	−0.50	−0.61	−0.65	−0.69	−0.75	−0.78	−0.78	−0.78			−0.78
2.60	−0.17	−0.37	−0.50	−0.60	−0.64	−0.68	−0.74	−0.76	−0.76	−0.77			−0.77
2.65	−0.18	−0.37	−0.50	−0.60	−0.64	−0.67	−0.73	−0.75	−0.75	−0.75			−0.75
2.70	−0.18	−0.38	−0.50	−0.60	−0.63	−0.67	−0.72	−0.73	−0.73	−0.74			−0.74
2.75	−0.19	−0.38	−0.50	−0.59	−0.63	−0.66	−0.71	−0.72	−0.72	−0.72			−0.73
2.80	−0.20	−0.38	−0.50	−0.59	−0.62	−0.65	−0.70	−0.71	−0.71	−0.71			−0.71
2.85	−0.21	−0.39	−0.50	−0.59	−0.62	−0.64	−0.69	−0.70	−0.70	−0.70			−0.70
2.90	−0.21	−0.39	−0.50	−0.58	−0.61	−0.64	−0.67	−0.68	−0.68	−0.69			−0.69
2.95	−0.22	−0.40	−0.50	−0.58	−0.61	−0.63	−0.66	−0.67	−0.67	−0.68			−0.68
3.00	−0.23	−0.40	−0.50	−0.57	−0.60	−0.62	−0.65	−0.66	−0.66	−0.67			−0.67

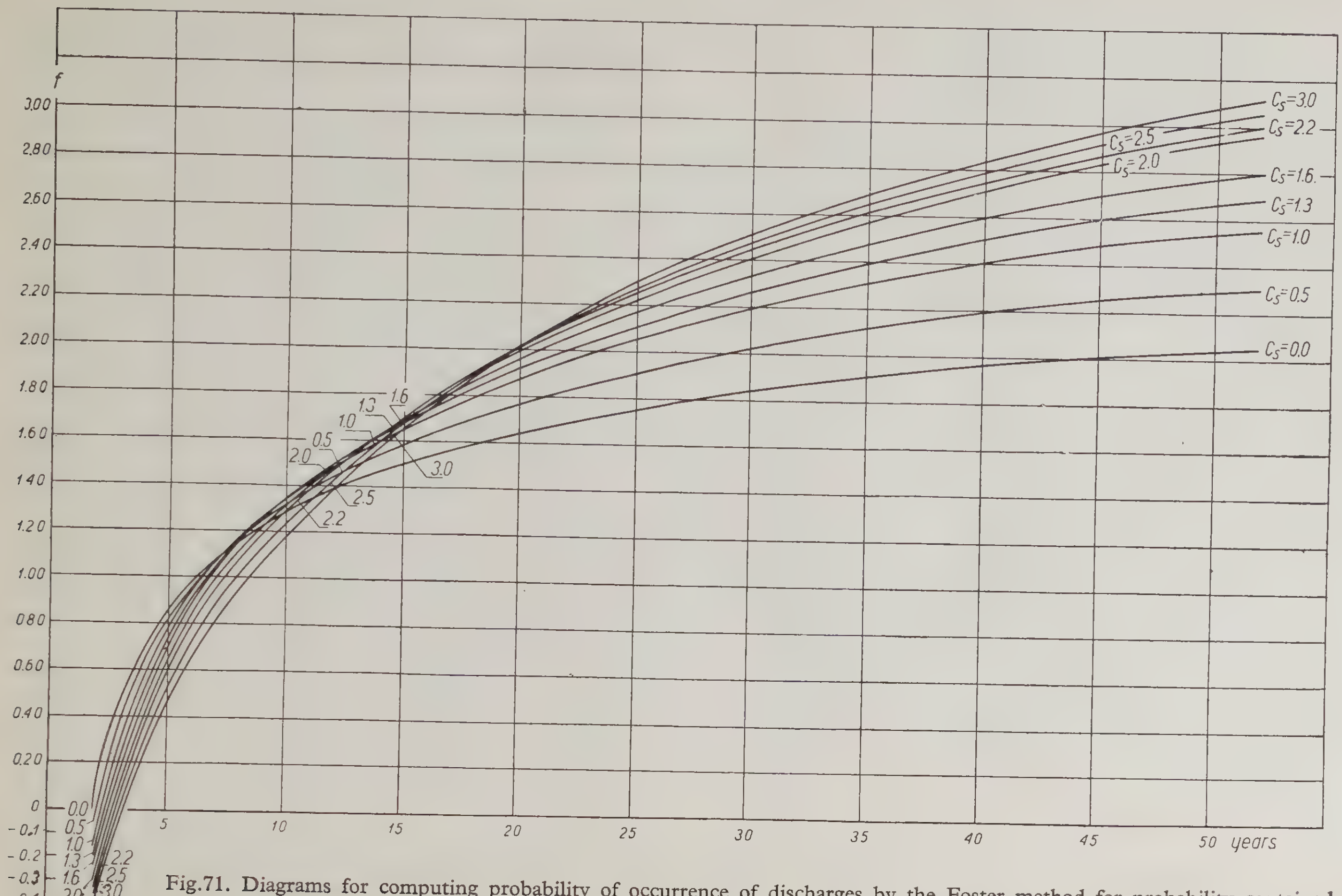


Fig.71. Diagrams for computing probability of occurrence of discharges by the Foster method for probability contained within the limits of 1 : 25 and 1 : 50 years

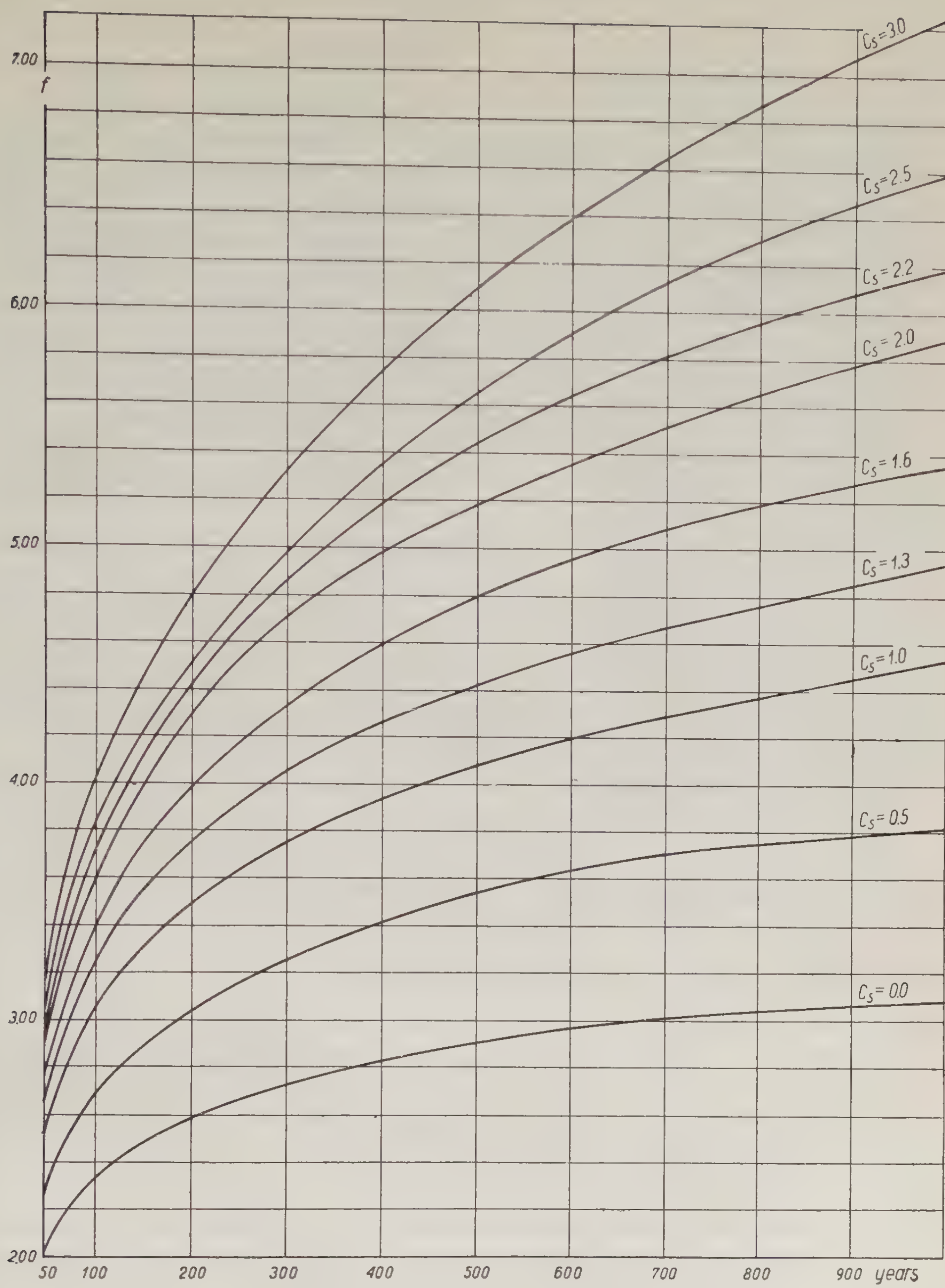


Fig. 72. Diagrams for computing probability of occurrence of discharges by the Foster method for probability contained within the limits of 1 : 50 and 1 : 1000 years

(9) drawing the theoretical integral curve of frequency by means of computed coefficient k, or direct maximum values of discharges Q, usually plotting the precentage of probability on the horizontal axis and maximum discharges, or coefficients k — on the vertical axis; an undulating line connecting these pionts is called a theoretical integral curve of frequency of discharges.

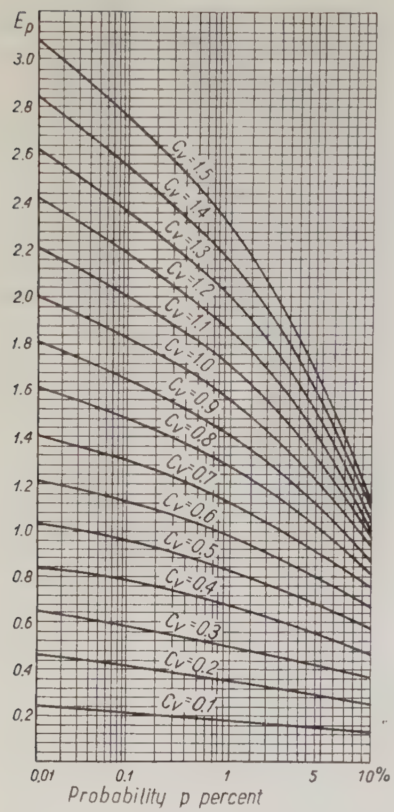


Fig. 73. Diagram for finding the value of an error.

The verification of computations consists in:

(a) adding up all values k, the sum of which is equal to the quantity of numbers of the series n,

(b) adding up all values $k - 1$, the sum of which should be equal to zero.

The discharges with various frequencies, computed for some Polish rivers by the Forster method are shown in Table 47.

A guaranteed correction of a maximum discharge for vital structures has been introduced by the Soviet regulations GOST 3999-48 effective since 1948.

A valid maximum discharge Q'_p, with a guaranteed correction ΔQ introduced in view of the inaccuracy of computations, is determined on the basis of the formula:

$$Q'_p = Q_p + \Delta Q$$

where Q_p — discharge at probability p without a guarantee correction.

A guarantee correction ΔQ is determined from the following correlation:

$$\Delta Q = \frac{aE_p}{\sqrt{n}} Q_p$$

where:

a — coefficient of value between 1 and 2; $a = 1$ is assumed for rivers sufficiently investigated and $a = 2$ for rivers inadequately investigated and having unstable discharge curves,

E_p — value of an error assumed from the diagram (Fig. 73),

n — quantity of numbers in a series (observations of maximum discharge).

Table 47

Discharges with Various Probabilities of Occurrence Computed by the Foster Method

No.	River	Cross section	Number of years of observation	Mean discharge Q_o	Coefficients of variation C_v	Coefficients of skewness C_s	Probability 1:5	1:10	1:20	1:25	1:33	1:50	1:100	1:200	1:300	1:600	1:1000
							Discharges in cu m/sec										
1	Vistula	Nowy Bieruń	52	206	0.44	1.50	269	327	383	401	423	453	508	560	589	647	680
2		Pustynia	54	287	0.45	1.50	377	459	539	565	596	640	717	792	832	915	962
3		Dwory	52	620	0.46	1.16	831	1002	1162	1216	1276	1359	1513	1655	1732	1886	1975
4		Tyniec	15	768	0.63	2.04	1058	1397	1736	1852	1987	2176	2520	2858	3052	3419	3657
5		Kraków	53	892	0.48	1.39	1196	1466	1723	1804	1907	2044	2288	2528	2656	2913	3067
6		Sandomierz	30	2317	0.36	1.30	2918	3435	3919	4077	4269	4527	4995	5437	5687	6162	6446
7		Puławy	20	2988	0.34	1.50	3699	4339	4969	5172	5416	5761	6371	6960	7275	7925	8301
8		Warsaw	54	3371	0.36	1.04	4281	4997	5652	5883	6114	6466	7072	7655	7970	8577	8941
9		Tczew	38	4260	0.32	1.26	5232	6077	6867	7126	7413	7835	8599	9317	9703	10466	10916
10	Wapiennica	Podkępie	8	72.5	0.61	1.77	101	131	160	170	181	197	226	255	270	301	320
11	Przemsza	Chełmek	20	81.9	0.38	1.35	104	124	142	148	155	165	183	200	209	227	238
12	Soła	Żywiec	59	348	0.53	1.41	480	595	706	743	785	844	953	1056	1112	1224	1289
13		Porąbka	26	467	0.62	1.63	661	849	1034	1095	1171	1272	1454	1628	1726	1918	2033
14		Oświęcim	22	614.8	0.74	1.85	906	1215	1516	1620	1743	1911	2216	2521	2680	3008	3208
15	Skawa	Wadowice	37	586.8	0.63	1.65	834	1075	1311	1393	1485	1618	1851	2077	2199	2450	2598
16	Rudawa	Balice	14	48.7	0.68	2.09	68.6	91.4	115	123	132	146	170	193	206	232	249
17	Raba	Stróże	20	262	0.99	2.08	418	597	781	843	918	1019	1206	1390	1494	1694	1826
18		Proszówki	22	628	0.61	1.49	896	1137	1375	1452	1544	1674	1900	2122	2245	2482	2628
19	Dunajec	Krościenko	22	588	0.77	1.93	869	1177	1494	1593	1720	1887	2204	2512	2680	3010	3219
20		Nowy Sącz	22	807.6	0.85	1.95	1233	1700	2181	2332	2524	2785	3265	3739	3993	4501	4817
21		Żabno	22	1344.6	0.69	1.52	1994	2579	3154	3349	3562	3887	4443	4991	5278	5872	6225
22	Nida	Brzegi	25	166	0.67	2.09	233	309	388	415	447	492	572	651	695	782	839
23	Wisłoka	Łabuzie	50	706	0.67	1.69	1018	1330	1638	1737	1860	2030	2333	2626	2787	3109	3303

Table 47 (continued)

No.	River	Cross section	Number of years of observation	Mean discharge Q_o	Coefficients: of variation C_v	Coefficients: of skewness C_s	Probability 1:5	1:10	1:20	1:25	1:33	1:50	1:100	1:200	1:300	1:600	1:1000
							Discharges in cu m/sec										
24	San	Korzeniów	50	833	0.59	1.34	1187	1492	1782	1875	1993	2145	2420	2691	2833	3123	3295
25		Żółtków	54	202	0.65	1.48	294	377	458	484	516	559	638	713	755	836	885
26		Gawłuszowice	49	1179	0.59	1.62	1652	2104	2542	2688	2862	3113	3544	3941	4198	4657	4935
27		Solina	12	769	0.47	1.96	993	1239	1492	1571	1673	1810	2063	2316	2446	2717	2883
28		Olchowce	12	741.5	0.57	1.85	1012	1299	1578	1676	1832	1946	2229	2512	2660	2965	3151
29		Przemyśl	82	1140	0.51	1.26	1559	1919	2256	2367	2489	2669	2995	3297	3466	3791	3983
30		Jarosław	49	1079	0.58	1.42	1523	1918	2293	2418	2562	2769	3132	3488	3676	4058	4283
31		Radomyśl	22	1748	0.59	1.98	2377	3089	3811	4048	4337	4729	5430	6172	6554	7327	7812
32	Iłżanka	Ciepielów	10	76.3	0.72	2.28	107	146	187	200	217	239	282	323	347	392	423
33	Wieprz	Krasnystaw	14	92	0.57	1.51	129	162	194	205	217	235	267	298	314	347	367
34		Lubartów	21	136	0.87	2.26	203	286	374	403	439	487	577	666	717	813	879
35		Kośmin	19	204	0.72	1.81	298	398	495	527	565	621	718	815	866	971	1034
36	Pilica	Szczekociny	21	23	0.72	1.69	33.9	44.9	55.6	59.1	63.4	69.4	80.0	90.2	95.9	107	114
37		Sulejów	25	170	0.55	1.85	230	293	353	377	402	436	499	562	594	662	703
38		Tomaszów Mazowiecki	29	199	0.56	2.06	266	344	422	449	480	524	604	682	726	812	868
39		Nowe Miasto	24	266	0.62	1.96	368	480	596	632	678	741	856	972	1031	1155	1231
40		Warka	28	360	0.65	1.85	510	669	823	877	940	1027	1184	1340	1422	1591	1694
41	Świder	Świder	15	91.3	0.51	1.38	124	154	182	190	202	216	243	269	283	311	327
42	Bug	Dorohusk	55	245	0.72	2.00	353	474	598	638	688	757	880	1004	1071	1203	1288
43		Brześć	54	411	0.69	1.94	587	780	978	1041	1120	1228	1423	1619	1724	1934	2064
44		Frankopol	54	466	0.69	2.00	662	884	1109	1183	1273	1398	1624	1849	1971	2212	2366
45		Wyszków	57	673	0.51	1.54	910	1129	1346	1414	1497	1617	1823	2025	2135	2358	2489
46		Zagrze	57	1087	0.52	1.76	1454	1833	2206	2325	2472	2681	3054	3416	3608	4004	4241

Table 47 (continued)

No.	River	Cross section	Number of years of observation	Mean discharge Q_o	Coefficients of variation C_v	Coefficients of skewness C_s	1:5	1:10	1:20	1:25	1:33	1:50	1:100	1:200	1:300	1:600	1:1000
							Discharges in cu m/sec										
47	Liwiec	Liwskie Mosty	32	100	0.85	1.93	153	210	270	289	312	344	403	461	493	555	594
48	Narew	Narew	17	71	0.24	1.50	82.9	93.7	104	108	112	118	128	138	143	154	160
49		Strękowa Góra	15	185	0.42	1.00	244	289	331	345	360	382	420	456	476	514	537
50		Piątnica	63	348	0.39	1.34	446	530	610	636	668	710	786	861	900	980	1028
51		Ostrołęka	63	438	0.48	1.50	585	718	848	890	940	1012	1138	1260	1325	1460	1538
52		Pułtusk	60	507	0.52	1.27	697	860	1013	1063	1121	1203	1348	1488	1564	1712	1801
53	Koszarawa	Swinna	25	175.8	0.34	1.48	218	255	292	304	319	338	374	408	427	464	487
54	Stradomka	Łapanów	32	162	0.56	1.22	228	284	335	353	372	400	449	496	521	571	601
55	Poprad	Muszyna	48	220	0.74	1.74	326	435	542	577	619	677	783	887	944	1057	1125
56		Milik	22	335	0.74	1.63	501	662	821	873	937	1024	1180	1329	1413	1577	1676
57		Stary Sącz	38	394	0.64	1.48	571	729	886	936	997	1080	1231	1375	1456	1612	1705
58	Biała	Koszyce	43	470.5	0.81	1.87	713	971	1221	1316	1411	1612	1809	2067	2200	2477	2643
59	Ropa	Gorlice	36	121	0.87	2.02	185	258	332	356	385	426	501	575	616	696	746
60	Solinka	Terka	22	208	0.43	1.20	273	328	379	396	415	442	490	535	560	610	638
61	Wiar	Krowniki	37	265	0.69	1.45	393	510	620	656	700	761	868	973	1029	1141	1209
62	Wisłok	Rzeszów	48	484	0.66	1.43	708	912	1104	1168	1244	1346	1522	1717	1813	2008	2126
63		Tryńcza	29	639	0.54	1.29	887	1101	1302	1367	1446	1553	1743	1926	2030	2226	2344
64	Tanew	Ulanów	34	112	0.64	1.64	160	207	252	268	286	311	356	400	425	473	501
65	Bystrzyca	Sobianowice	14	60.7	0.28	1.22	73.1	83.5	93.2	96.4	100	105	114	123	128	137	143
66	Krzna	Malowa Góra	27	106	0.72	1.97	153	205	259	276	297	327	380	433	457	511	554
67	Biebrza	Burzyn	12	283	0.65	1.61	408	528	644	682	728	793	908	1011	1080	1201	1273
68	Pissa	Ptaki	18	116	0.38	1.63	146	174	202	212	223	239	266	293	308	337	354
69	Wkra	Cieksyn	14	147	0.48	1.44	196	242	284	298	315	338	379	419	441	484	510

The computation of the discharge Q_p should be checked if the guaranteed correction ΔQ results in a number higher than 0.2 Q_p.

Determining the Probability Curve by means of Empirical Data

To check the theoretical curve of probability of maximum discharges, the values of maximum discharges observed should be plotted on the diagram with the corresponding percentage probability computed by the following three formulas:

$$p = \frac{m - 0.5}{n} 100$$

$$p = \frac{m}{n + 1} 100$$

$$p = \frac{m - 0.3}{n + 0.4} 100$$

where:

p — percentage probability sought,

m — successive number of a discharge in a distributive series (the discharges being arranged in decreasing order),

n — total number of observations (numbers of a series).

A correctly drawn curve of frequency should pass through the points, or between the points computed by the formulas shown above. In the event of considerable discrepancies the computations of Q_o, C_v and C_s should be checked. All these formulas usually yield identical values p in the central part of the curve of frequency of maximum discharges, while the greatest differences mostly occur on the ends of the curve.

Let us investigate how many numbers of the series yield the 1 percent probability once in a 100 years — if the formulas indicated above are applied:

$$p = \frac{m - 0.5}{n} 100 = \frac{1 - 0.5}{50} 100 = 1\%$$

$$p = \frac{m}{n + 1} 100 = \frac{1}{99 + 1} 100 = 1\%$$

$$p = \frac{m - 0.3}{n + 0.4} 100 = \frac{1 - 0.3}{70 + 0.4} 100 = 1\%$$

These computations show that — using the first formula — a discharge with a once-in-100 years frequency is obtained from 50 observations, from the second formula — 99 observations and from the third 70 observations.

The most appropriate formula is that which yields frequencies nearest to the period of observations. In this respect, the best results are obtained from

the second formula; the worst are from the first, according to which, with a series consisting of 50 numbers, the first of them has the once-in-100 years frequency.

For this reason, the use of the second formula, accepted by GOST 3999-48, has been introduced in the Soviet Union to replace the first formula, formerly used for computing the percentage probability. But even this formula does not yield satisfactory results, because the points computed by it for the low percentage of probability bend the curve of probability to the right, which causes an unfounded increase in maximum discharges, particularly with high coefficients of variation and skewness.

The use of the third formula, therefore, yielding values contained between those computed by the first and second formulas was recently started in the Soviet Union.

The transition from the probability p percent to the probability N years for the sector of the curve for wet years (up to 50 percent of probability) is achieved by means of the formula:

$$N = \frac{100}{p}$$

The probability for dry years — i. e., for probability above 50 percent — is determined in practice by one of the following formulas:

$$N = \frac{100 - p}{100} \text{ or } N = \frac{100}{100 - p}$$

Table 48

Correlation between the Probability of Discharges

Probability p percent	Probability once in N years			Characteristics of a year
	wet years	dry years		
	$N = \frac{100}{p}$	$N = \frac{100}{100 - p}$	$N = \frac{100 - p}{100}$	
0.1	1000			disastrously wet
1	100			very wet
3	33			wet
5	20			wet
10	10			average wet
25	4			slightly wet
50	2			average
75		4	0.25	slightly dry
90		10	0.10	average dry
95		20	0.05	dry
97		33	0.03	dry
99		100	0.01	very dry
99.9		1000	0.001	disastrous droughts

Discharges whose probability were recomputed by the latter formulas are presented in Table 48.

Determining the Integral Probability Curve in the Scale of Probability

The upper and lower parts of the integral curve of probability drawn to the usual scale, are steep. For this reason, the accuracy of discharges obtained by extrapolation in the upper and lower ends of this curve is low, although designers are usually most interested in these parts of a curve.

Therefore, to increase accuracy in determining discharges, the integral curve of probability is drawn in the scale of probability, which allows for a decrease in the curvature of this curve until it becomes completely straight.

The phenomenon of the normal Gauss curve of frequency (at $C_s = 0$) plotted in this scale, assuming the shape of a straight line, is the principal property of the scale of probability. A curve with convexity turned downwards is obtained for the positive coefficient of skewness, while for the negative coefficient, it is pointed upwards.

Ready forms with this scale plotted on them (Fig. 74) are usually applied in drawing diagrams in the scale of probability. If such forms are not available, the scale of probability may be plotted according to Table 49, where the distances are shown from the center of the scale of probability (50 percent). Since this scale is symmetric to the center, the distances presented in the Table are plotted on both sides of the center of the scale.

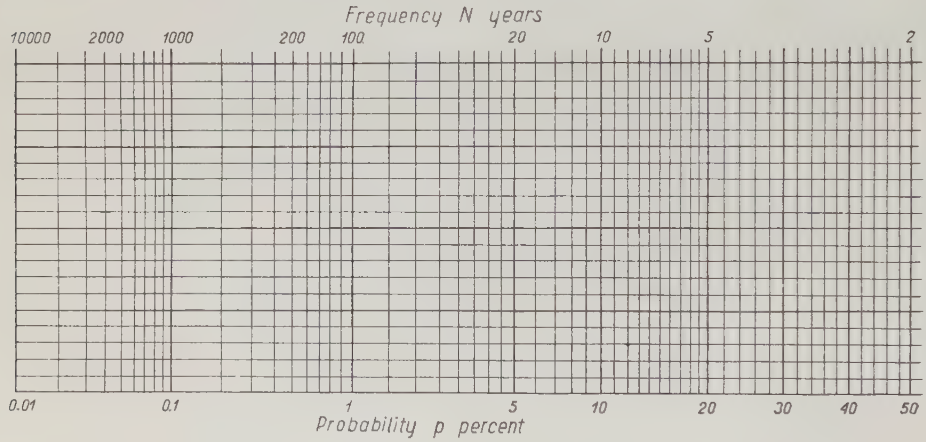


Fig. 74. Scale of probability

A normal vertical graduation of the scale is adopted for determining integral curves of probability having an insignificant skewness asymmetry $C_s < 2\ C_v$, while a logarithmic vertical graduation (Table 50) must be used in case of great skewness $C_s > 2\ C_v$.

Table 49

Data for Drawing the Scale of Probability

Percentage probability p	Horizontal distance from the center 50% in mm	Percentage probability p	Horizontal distance from the center 50% in mm	Percentage probability p	Horizontal distance from the center (50%) in mm
50	0	16	42.7	1.0	100.0
48	2.1	15	44.6	0.9	101.7
46	4.3	14	46.5	0.8	103.6
44	6.5	13	48.5	0.7	105.7
42	8.7	12	50.5	0.6	108.1
40	10.9	11	52.7	0.5	110.8
38	13.1	10	55.1	0.4	114.0
36	15.4	9	57.6	0.3	118.2
34	17.7	8	60.4	0.2	123.7
32	20.1	7	63.5	0.1	133.0
30	22.5	6	66.8	0.09	134.2
28	25.0	5	70.7	0.08	135.8
26	27.6	4.5	72.9	0.07	137.7
24	30.3	4.0	75.3	0.06	139.7
22	33.2	3.5	78.0	0.05	142.1
20	36.2	3.0	80.9	0.04	145.0
19	37.7	2.5	84.3	0.03	148.5
18	39.3	2.0	88.3	0.02	153.4
17	41.0	1.5	93.3	0.01	161.0

The integral curve of probability in the event of the existence of several discharges observed and computed by Q_o, C_v and C_s, is drawn in the scale of probability by the method used in drawing it to a usual scale.

Drawing the curve (without computing Q_o, C_v and C_s) in the scale of probability, consists in plotting discharges observed according to decreasing order at corresponding percentage probability, computed from the formula:

$$p = \frac{m - 0.3}{n + 0.4} 100$$

An undulating line is drawn through the points thus obtained. The upper and lower part of the curve outside the limits of observation are drawn by extrapolation; the general direction of the curve, however, must be taken into account.

The results of computing probability of the discharge occurrence on the River Biała in Koszyce are collected in Table 51. The integral curve of probability drawn by means of these computations is presented in Fig. 75.

The distributive series of the maximum discharges on the Biała River in Koszyce give the following parameters: —

— coefficient of variation of the series:

Values for Drawing the Logarithmic Graduation

Discharge in cu m/sec numbers computed by logarithms	Distances from the beginning of the system of coordinates in cm	Discharge in cu m/sec numbers computed by logarithms	Distances from the beginning of the system of coordinates in cm
1	0.00	800	14.52
5	3.50	900	14.77
10	5.00	1000	15.00
20	6.51	2000	16.51
30	7.39	3000	17.39
40	8.01	4000	18.01
50	8.50	5000	18.50
60	8.89	6000	18.89
70	9.23	7000	19.23
80	9.52	8000	19.52
90	9.77	9000	19.77
100	10.00	10000	20.00
150	10.88	20000	21.51
200	11.51	30000	22.39
250	11.99	40000	23.01
300	12.39	50000	23.50
350	12.72	60000	23.89
400	13.01	70000	24.23
450	13.27	80000	24.52
500	13.50	90000	24.77
600	13.89	100000	25.00
700	14.23		

$$C_v = \sqrt{\frac{\sum (k-1)^2}{n}} = \sqrt{\frac{27.9636}{43}} = 0.806$$

— coefficient of skewness of the series:

$$C_s = \frac{2\,C_v}{1 - k_{min}} = \frac{2 \times 0.806}{1 - 0.14} = \frac{1.612}{0.86} = 1.87$$

The following values of maximum discharges with various probabilities are computed by the Foster formula $Q_p = Q_o\,(fC_v + 1)$:

$Q_{0.1\%} = 470.49\ (0.806 \times 5.73 + 1) = 2643.4$ cu m/sec
$Q_{1\%} = 470.49\ (0.806 \times 3.53 + 1) = 1809.1$ „ „
$Q_{2\%} = 470.49\ (0.806 \times 3.01 + 1) = 1611.9$ „ „
$Q_{4\%} = 470.49\ (0.806 \times 2.23 + 1) = 1316.1$ „ „
$Q_{10\%} = 470.49\ (0.806 \times 1.32 + 1) = 971.1$ „ „

Table 51

Computation of the Distributive Series of Maximum Discharges Q_{max} on the River Biała in Koszyce

No.	Year	Q_{max}	k	$k-1$	$(k-1)^2$	$\frac{m-0.3}{n+0.4}100$
1	1901	1635	3.48	+2.48	6.1504	1.6129
2	1934	1635	3.48	+2.48	6.1504	3.9170
3	1909	1282	2.72	+1.72	2.9584	6.2212
4	1931	1103	2.34	+1.34	1.7956	8.5253
5	1918	856	1.82	+0.82	0.6724	10.8295
6	1920	804	1.71	+0.71	0.5041	13.1336
7	1913	751	1.60	+0.60	0.3600	15.4378
8	1916	751	1.60	+0.60	0.3600	17.7419
9	1926	751	1.60	+0.60	0.3600	20.0461
10	1903	725	1.54	+0.54	0.2916	22.3502
11	1908	725	1.54	+0.54	0.2916	24.6544
12	1906	605	1.29	+0.29	0.0841	26.9585
13	1948	586	1.25	+0.25	0.0625	29.2627
14	1951	560	1.19	+0.19	0.0361	31.5768
15	1899	477	1.01	+0.01	0.0001	33.8710
16	1898	440	0.94	−0.06	0.0036	36.1751
17	1929	423	0.90	−0.10	0.0100	38.4793
18	1941	420	0.89	−0.11	0.0121	40.7834
19	1927	405	0.86	−0.14	0.0196	43.0876
20	1936	400	0.85	−0.15	0.0225	45.3917
21	1900	391	0.83	−0.17	0.0289	47.6959
22	1943	391	0.83	−0.17	0.0289	50.0000
23	1928	374	0.79	−0.21	0.0441	52.3041
24	1902	320	0.68	−0.32	0.1024	54.6083
25	1912	317	0.67	−0.33	0.1089	56.9124
26	1905	292	0.62	−0.38	0.1444	59.2166
27	1907	286	0.61	−0.39	0.1521	61.5207
28	1922	236	0.50	−0.50	0.2500	63.8249
29	1949	213	0.45	−0.55	0.3025	66.1290
30	1911	207	0.44	−0.56	0.3136	68.4332
31	1904	195	0.41	−0.59	0.3481	70.7373
32	1937	185	0.39	−0.61	0.3721	73.0415
33	1930	170	0.36	−0.64	0.4096	75.3456
34	1932	164	0.35	−0.65	0.4225	77.6498
35	1947	164	0.35	−0.65	0.4225	79.9539
36	1935	159	0.34	−0.66	0.4356	82.2581
37	1921	146	0.31	−0.69	0.4761	84.5622
38	1942	142	0.30	−0.70	0.4900	86.8664
39	1933	139	0.30	−0.70	0.4900	86.1705
40	1910	126	0.27	−0.73	0.5329	91.4747
41	1917	122	0.26	−0.74	0.5476	93.7788
42	1946	90	0.19	−0.81	0.6561	96.0830
43	1950	68	0.14	−0.86	0.7396	98.3871
Total		20231	43.00	+13.17 −13.17	27.9636	
Mean		470.49				

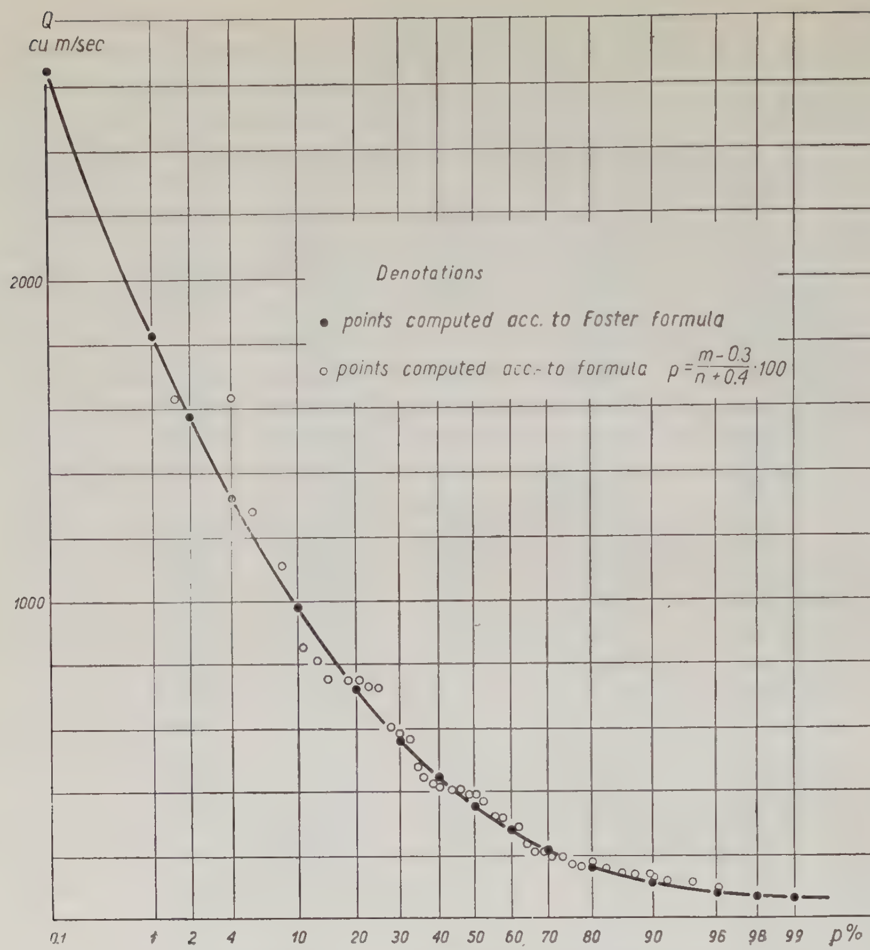


Fig. 75. The integral probability curve of discharges for the Koszyce cross section on the Biała River

$$Q_{20\%} = 470.49\ (0.806 \times 0.64 + 1) = 713.2 \text{ cu m/sec}$$
$$Q_{30\%} = 470.49\ (0.806 \times 0.23 + 1) = 557.7 \text{ ,, ,,}$$
$$Q_{40\%} = 470.49\ [0.806 \times (-0.06) + 1] = 447.7 \text{ ,, ,,}$$
$$Q_{50\%} = 470.49\ [0.806 \times (-0.28) + 1] = 364.3 \text{ ,, ,,}$$
$$Q_{60\%} = 470.49\ [0.806 \times (-0.48) + 1] = 288.5 \text{ ,, ,,}$$
$$Q_{70\%} = 470.49\ [0.806 \times (-0.64) + 1] = 227.8 \text{ ,, ,,}$$
$$Q_{80\%} = 470.49\ [0.806 \times (-0.80) + 1] = 167.1 \text{ ,, ,,}$$
$$Q_{90\%} = 470.49\ [0.806 \times (-0.93) + 1] = 117.8 \text{ ,, ,,}$$
$$Q_{96\%} = 470.49\ [0.806 \times (-1.01) + 1] = 87.5 \text{ ,, ,,}$$

$$Q_{98\%} = 470.49\ [0.806 \times (-1.04) + 1] = \quad 76.1 \text{ cu m/sec}$$
$$Q_{99\%} = 470.49\ [0.806 \times (-1.05) + 1] = \quad 72.3 \quad ,, \quad ,,$$
$$Q_{99.9\%} = 470.49\ [0.806 \times (-1.07) + 1] = \quad 64.7 \quad ,, \quad ,,$$

4. Computing Maximum Water Stages with Various Probabilities of Occurrence

When accepting the height of the lower structure of a bridge — or for other purposes — the water level H should be determined at a reliable discharge. This level can best be determined on the basis of the discharge curve denoted by the equation $Q = f(H)$.

The discharges with very low probability of occurrence are, however usually not included in the upper branch of this curve. If the multiannual observations of the highest water levels are available for the cross section under study, the level corresponding with the discharge of a defined frequency may be determined by the integral curve of frequency of water levels.

The theory of probability has not hitherto been very widely used for computing maximum water stages with various frequencies, on account of the instability of parameters of the integral curve of frequency of such stages. It is known that the coefficient of variation and the mean value of a series of maximum stages vary in correlation with an assumed zero stage of a water gage, highest water level, etc.

It was determined however, from detailed computations, that the coefficient of skewness remains constant, irrespective of a change in the coefficient of variation and mean water stage for a change in the zero stage of a water gage from which the levels of a series under study are determined. For this reason, the water stage with frequency thus determined will also remain constant.

Coefficients of skewness C_s of the integral curves of probability of maximum water stages in rivers can have positive or negative values, mostly fluctuating within limits of $+1.4$ and -1.2.

The integral curves of water stage probability with positive asymmetry drawn in the scale of probability have their bends turned downwards, while those with negative asymmetry — upwards. The correlation of maximum stages with the percentage probability is, in the event of the asymmetry being equal to zero, represented by a straight line.

The occurrence of the integral curve of probability of maximum water stages with negative asymmetry can be explained by the fact that the intensity of increase in the maximum water stages considerably diminishes as the water stages reach their highest magnitudes. In such cases, water spreads over the valley flat, considerably increasing the size of the transverse section of a river.

A sufficiently long decreasing series of maximum stages with negative asymmetry usually has the value of a median higher than one.

Empirical Method of Determining the Integral Probability Curve for Maximum Water Stages

A table of maximum stages is first prepared for a multiannual period, whose numbers are arranged in decreasing order. A percentage probability for each stage is computed from the formula:

$$p = \frac{m - 0.3}{n + 0.4} 100$$

The value of maximum stages for probability thus computed is plotted on the graph in the scale of probability with the axis of ordinates drawn according to the usual scale.

The integral curve of probability is drawn through points thus obtained, and its end parts are approximately determined by extrapolation with consideration of the direction of the curve.

Ordinate Values of Deviations of the Integral Curve of Probability

C_s \ %	0.1	0.3	0.5	1	3	5	10	20	25	30	40
−0.0	3.09	2.75	2.58	2.33	1.88	1.64	1.28	0.84	0.67	0.52	0.25
−0.1	2.95	2.64	2.48	2.25	1.84	1.61	1.27	0.85	0.68	0.53	0.27
−0.2	2.81	2.53	2.39	2.18	1.79	1.58	1.26	0.85	0.69	0.55	0.28
−0.3	2.67	2.42	2.29	2.10	1.75	1.55	1.24	0.85	0.70	0.56	0.30
−0.4	2.54	2.31	2.20	2.03	1.70	1.52	1.23	0.85	0.71	0.57	0.31
−0.5	2.40	2.21	2.11	1.96	1.66	1.49	1.22	0.85	0.71	0.58	0.33
−0.6	2.27	2.10	2.02	1.88	1.61	1.45	1.20	0.85	0.72	0.59	0.34
−0.7	2.14	2.00	1.93	1.81	1.57	1.42	1.18	0.85	0.72	0.60	0.36
−0.8	2.02	1.90	1.84	1.74	1.52	1.38	1.17	0.85	0.73	0.60	0.37
−0.9	1.90	1.80	1.75	1.66	1.47	1.35	1.15	0.85	0.73	0.61	0.38
−1.0	1.79	1.71	1.66	1.59	1.42	1.32	1.13	0.85	0.73	0.62	0.39
−1.1	1.68	1.62	1.58	1.52	1.38	1.28	1.10	0.85	0.74	0.62	0.41
−1.2	1.58	1.53	1.50	1.45	1.33	1.24	1.08	0.84	0.74	0.63	0.42
−1.3	1.48	1.45	1.42	1.38	1.28	1.20	1.06	0.84	0.74	0.63	0.43
−1.4	1.39	1.37	1.35	1.32	1.23	1.17	1.04	0.83	0.73	0.64	0.44
−1.5	1.31	1.30	1.28	1.26	1.19	1.13	1.02	0.82	0.73	0.64	0.45
−1.6	1.24	1.23	1.22	1.20	1.14	1.10	0.99	0.81	0.73	0.64	0.46
−1.7	1.17	1.16	1.15	1.14	1.10	1.06	0.97	0.81	0.72	0.64	0.47
−1.8	1.11	1.10	1.10	1.09	1.06	1.02	0.94	0.80	0.72	0.64	0.48
−1.9	1.05	1.05	1.04	1.04	1.01	0.98	0.92	0.79	0.72	0.64	0.48
−2.0	0.999	0.997	0.995	0.99	0.97	0.95	0.90	0.78	0.71	0.64	0.49

The valid values of water stages with low probability of occurrence are read from a curve thus drawn.

Analytical Method of Determining the Integral Probability Curve for Maximum Water Stages

Applying the analytical method of determining the integral curve of probability of water stages, the parameters of this curve should be primarily computed by the following equations:

$$H_o = \frac{\sum H}{n}$$

$$C_v = \sqrt{\frac{\sum (k-1)^2}{n-1}}$$

$$C_s = \frac{\sum (k-1)^3}{(n-1)\, C_v^3}$$

Table 52

from the Center at $C_v = 1.0$, and assuming Negative Asymmetry

50	60	70	75	80	90	95	97	99	99.5	99.7	99.9
0.00	−0.25	−0.52	−0.67	−0.84	−1.28	−1.64	−1.88	−2.33	−2.58	−2.75	−3.09
0.02	−0.24	−0.51	−0.66	−0.84	−1.29	−1.67	−1.92	−2.40	−2.67	−2.85	−3.23
0.03	−0.22	−0.50	−0.65	−0.83	−1.30	−1.70	−1.96	−2.47	−2.76	−2.96	−3.38
0.05	−0.20	−0.48	−0.64	−0.82	−1.31	−1.72	−2.00	−2.54	−2.86	−3.07	−3.52
0.07	−0.19	−0.47	−0.63	−0.82	−1.32	−1.75	−2.04	−2.61	−2.95	−3.18	−3.66
0.08	−0.17	−0.46	−0.62	−0.81	−1.32	−1.77	−2.08	−2.68	−3.04	−3.29	−3.81
0.10	−0.16	−0.44	−0.61	−0.80	−1.33	−1.80	−2.12	−2.75	−3.13	−3.40	−3.96
0.12	−0.14	−0.43	−0.59	−0.79	−1.33	−1.82	−2.15	−2.82	−3.22	−3.50	−4.10
0.13	−0.12	−0.41	−0.58	−0.78	−1.34	−1.84	−2.18	−2.89	−3.31	−3.61	−4.24
0.15	−0.11	−0.40	−0.57	−0.77	−1.34	−1.86	−2.22	−2.96	−3.40	−3.72	−4.38
0.16	−0.09	−0.38	−0.55	−0.76	−1.34	−1.88	−2.25	−3.02	−3.49	−3.82	−4.53
0.18	−0.07	−0.36	−0.54	−0.74	−1.34	−1.89	−2.28	−3.09	−3.58	−3.92	−4.67
0.19	−0.05	−0.35	−0.52	−0.73	−1.34	−1.91	−2.31	−3.15	−3.66	−4.03	−4.81
0.21	−0.04	−0.33	−0.51	−0.72	−1.34	−1.92	−2.34	−3.21	−3.74	−4.13	−4.95
0.23	−0.02	−0.31	−0.49	−0.71	−1.34	−1.94	−2.37	−3.27	−3.83	−4.23	−5.09
0.24	−0.00	−0.30	−0.47	−0.69	−1.33	−1.95	−2.39	−3.33	−3.91	−4.33	−5.23
0.25	−0.02	−0.28	−0.46	−0.68	−1.33	−1.96	−2.42	−3.39	−3.99	−4.42	−5.37
0.27	−0.03	−0.26	−0.44	−0.66	−1.32	−1.97	−2.44	−3.44	−4.07	−4.52	−5.50
0.28	−0.05	−0.24	−0.42	−0.64	−1.32	−1.98	−2.46	−3.50	−4.15	−4.62	−5.64
0.29	−0.07	−0.22	−0.40	−0.63	−1.31	−1.99	−2.49	−3.55	−4.23	−4.71	−5.77
0.31	−0.08	−0.20	−0.39	−0.61	−1.30	−2.00	−2.51	−3.60	−4.30	−4.81	−5.91

The integral curve of probability of maximum water stages is drawn on the basis of parameters thus computed, using tables of ordinate deviations f. At the same time, attention should be paid to the fact that the magnitude of ordinates f for positive values of C_s is established on the basis of the previously presented Foster table (Table 46), while corresponding numbers for negative values of C_s are determined from Table 52, which shows values f at $C_v = 1$ and negative asymmetry.

The integral curve of probability is drawn in the scale of probability on the basis of values thus obtained. If the integral curve of probability does not take a central position to points representing the highest water stages observed or does not pass through these points, all computations should be checked, because such deviations are evidence of errors.

Note that the Foster table (Table 46) may also be used for determining value f in the event of negative asymmetry. In such cases, probability for the complement should be changed to 100, and all ordinates of deviations should be adopted with an opposite sign.

If, for instance, it is necessary to compute the deviation of the ordinate f for the coefficient of skewness $C_s = -1$ and probability 3 percent, the value f is established from the column of probability $100 - 3 = 97$ percent, and from the line $C_s = 1$. Changing the sign of f into the opposite sign, we arrive at $f = 1.42$.

Table 53 shows the collected results of computations of probability of occurrence of maximum water stages on the River Mała Panew in Luboszyce. The integral curve of probability drawn by these computations is shown (Fig. 76).

The distributive series of maximum stages on the River Mała Panew in Luboszyce give the following parameters:

C_v — coefficient of variation of the series:

$$C_v = \sqrt{\frac{\sum (k-1)^2}{n}} = \sqrt{\frac{1.5662}{28}} = 0.237$$

C_s — coefficient of skewness of the series:

$$C_s = \frac{\sum (k-1)^3}{nC_v^3} = \frac{-0.0040}{28 \times 0.0133} = \frac{-0.0040}{0.3724} = -0.01$$

Maximum stages with various probability computed from the Foster formula $H_p = H_o (fC_v + 1)$ give the following values:

$$H_{0.1\%} = 204.36\ [(0.237 \times 3.08) + 1] = 353.5 \text{ cm}$$
$$H_{1\%} = 204.36\ [(0.237 \times 2.32) + 1] = 316.7 \text{ ,,}$$
$$H_{2\%} = 204.36\ [(0.237 \times 2.10) + 1] = 306.1 \text{ ,,}$$
$$H_{4\%} = 204.36\ [(0.237 \times 1.76) + 1] = 289.6 \text{ ,,}$$
$$H_{10\%} = 204.36\ [(0.237 \times 1.28) + 1] = 266.4 \text{ ,,}$$

Table 53

Computation of the Distributive Series of Maximum Water Stages H_{max} on the River Mała Panew in Luboszyce

No.	H_{max} cm	k	$k-1$	$(k-1)^2$	$(k-1)^3$	$\frac{m-0.3}{n+0.4}100$
1	291	1.42	+0.42	0.1764	+0.0741	2.4648
2	284	1.39	+0.39	0.1521	+0.0593	5.9859
3	279	1.37	+0.37	0.1369	+0.0507	9.5070
4	261	1.28	+0.28	0.0784	+0.0220	13.0282
5	250	1.22	+0.22	0.0484	+0.0106	16.5493
6	246	1.20	+0.20	0.0400	+0.0080	20.0704
7	241	1.18	+0.18	0.0324	+0.0058	23.5915
8	241	1.18	+0.18	0.0324	+0.0058	27.1127
9	239	1.17	+0.17	0.0289	+0.0049	30.6338
10	236	1.15	+0.15	0.0225	+0.0034	34.1549
11	236	1.15	+0.15	0.0225	+0.0034	37.6761
12	210	1.03	+0.03	0.0009	0.0000	41.1972
13	208	1.02	+0.02	0.0004	0.0000	44.7183
14	206	1.01	+0.01	0.0001	0.0000	48.2394
15	204	1.00	0.00	0.0000	0.0000	51.7606
16	203	0.99	−0.01	0.0001	0.0000	55.2817
17	200	0.98	−0.02	0.0004	0.0000	58.8028
18	186	0.91	−0.09	0.0081	−0.0007	62.3239
19	176	0.86	−0.14	0.0196	−0.0027	65.8451
20	175	0.86	−0.14	0.0196	−0.0027	69.3662
21	171	0.84	−0.16	0.0256	−0.0041	72.8873
22	157	0.77	−0.23	0.0529	−0.0122	76.4085
23	154	0.75	−0.25	0.0625	−0.0156	79.9296
24	144	0.70	−0.30	0.0900	−0.0270	83.4507
25	137	0.67	−0.33	0.1089	−0.0359	86.9718
26	134	0.66	−0.34	0.1156	−0.0393	90.4930
27	132	0.65	−0.35	0.1225	−0.0429	94.0141
28	121	0.59	−0.41	0.1681	−0.0689	97.5352
Total	5722	28.00	+2.77 −2.77	1.5662	+0.2480 −0.2520	
Mean	204.36				−0.0040	

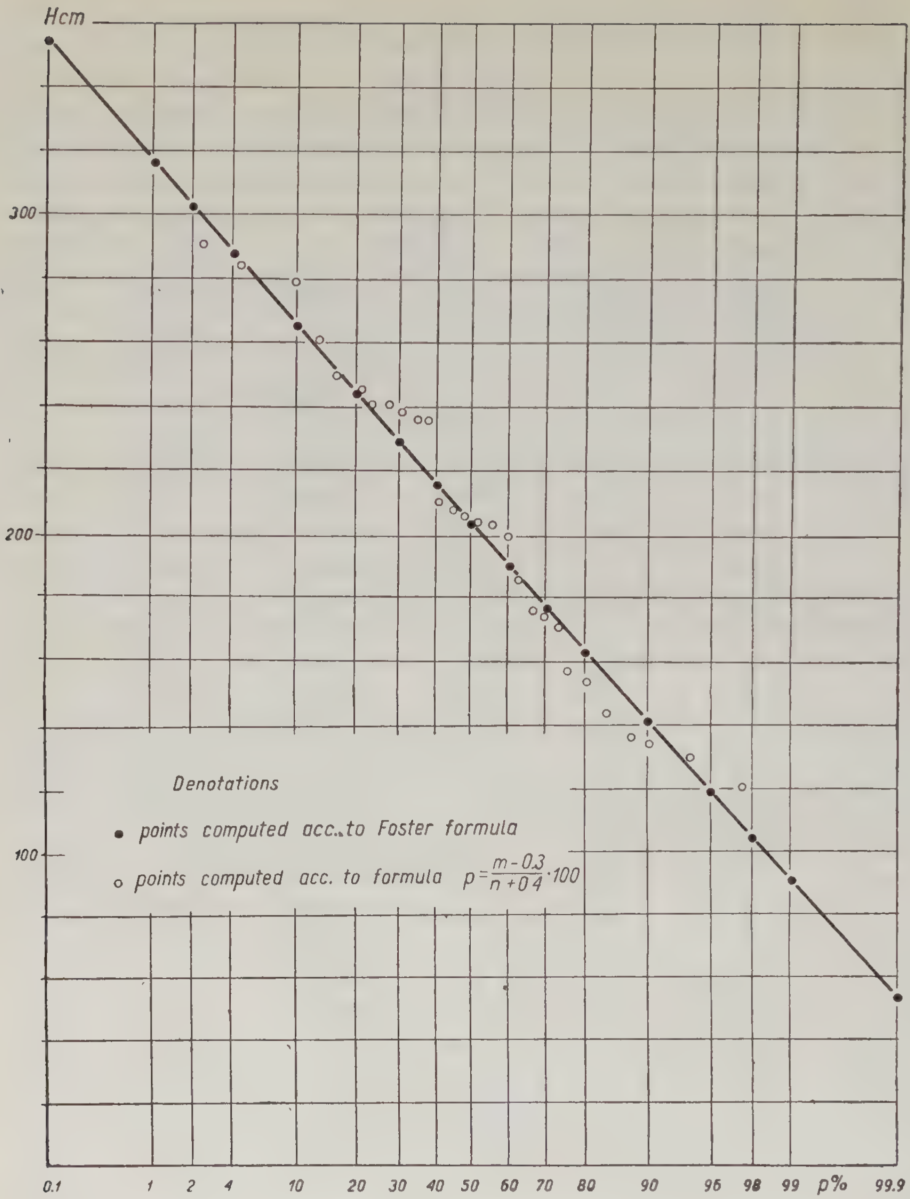


Fig. 76. The integral probability curve of the maximum water levels in Luboszyce on the Mała Panew River

$$H_{20\%} = 204.36\ [(0.237 \times 0.84) + 1] = 245{,}0 \text{ cm}$$
$$H_{30\%} = 204.36\ [(0.237 \times 0.52) + 1] = 229.5 \ ,,$$
$$H_{40\%} = 204.36\ [(0.237 \times 0.25) + 1] = 216.5 \ ,,$$

$$H_{50\%} = 204.36\ [(0.237 \times \quad 0.00) + 1] = 204.4 \text{ cm}$$
$$H_{60\%} = 204.36\ [\ 0.237 \times (-0.25) + 1] = 192.3 \quad ,,$$
$$H_{70\%} = 204.36\ [\ 0.237 \times (-0.52) + 1] = 179.2 \quad ,,$$
$$H_{80\%} = 204.36\ [\ 0.237 \times (-0.84) + 1] = 163.7 \quad ,,$$
$$H_{90\%} = 204.36\ [\ 0.237 \times (-1.28) + 1] = 142.4 \quad ,,$$
$$H_{96\%} = 204.36\ [\ 0.237 \times (-1.76) + 1] = 119.1 \quad ,,$$
$$H_{98\%} = 204.36\ [\ 0.237 \times (-2.11) + 1] = 102.2 \quad ,,$$
$$H_{99\%} = 204.36\ [\ 0.237 \times (-2.34) + 1] = \ 91.0 \quad ,,$$
$$H_{99.6\%} = 204.36\ [\ 0.237 \times (-3.10) + 1] = \ 54.2 \quad ,,$$

5. Determining Probability of Discharges for Short Distributive Series or in Absence of Observations

The extrapolation of the integral curve of frequency of discharges is most accurate when observation data from a long period are available; therefore every effort should always be made to increase the quantity of numbers in a series and not to discard the values of discharges observed, when it is proved that these magnitudes are reliable.

Determining the probability of rises in the event of a complete absence of observations is the most difficult task.

Computing Discharges with Various Probabilities by Short Series of Observations

For some cross sections, we know the result of observations conducted for a short period and, at the same time, we know a very high rise, which appeared before the systematic observations were put in operation. The water level and data of such a rise are sometimes marked on the bridge pillars, or can be indicated by local residents.

The Krytski and Menkel method, and the Boldakov method, are generally used for including such rises in a distributive series.

The Krytski - Menkel Method

If we know an uninterrupted series of maximum discharges over a short period of n years and one discharge Q_N which is known as a maximum during the period of N years longer than n, the parameters Q_o and C_v of the integral curve of frequency can be computed by the formulas presented by Krytski and Menkel:

$$Q_o = \frac{1}{N}\left(Q_N + \frac{N-1}{n} \sum_{1}^{n} Q\right)$$

$$C_v = \sqrt{\frac{1}{N}\left[(K_N - 1)^2 + \frac{N-1}{n}\sum_1^n (K-1)^2\right]}$$

where:

$\sum_1^n Q$ — sum of discharges of an uninterrupted series of observations during the period n,

K and K_N — coefficients of modulus computed from the formulas:

$$K = \frac{Q_i}{Q_o} \text{ and } K_N = \frac{Q_N}{Q_o}$$

$\sum^n (k-1)^2$ — sum of n numbers for an uninterrupted series, according to Foster.

The coefficient of skewness C_s is computed by means of one of the following formulas:

$$C_s = \frac{2\,C_v}{1 - K_{min}}$$

or: $C_s = 2C_v$ for spring rises, and
$C_s = 4C_v$ for summer rises.

The Boldakov Method

Three methods of including maximum discharges in a short period of observations have been presented by Boldakov.

The first of these consists in placing the known value of a disastrous discharge in a series and increasing the quantity of numbers in a series by filling the interval between the short series of observations and the separately appearing disastrous discharge. The interval is filled by ordinarly writing down the successive values of discharges from a short period of known observations.

In 1938 Boldakov presented the second method, described below, of introducing the separately occurring disastrous discharges into a short period of observations. This method involves the following order of procedure:

(1) if a short — for instance a 10-year — series of maximum annual discharges is available, the known disastrous rise Q_k not appearing in such series is adopted as an eleventh number of the series;
(2) a mean annual maximum discharge is computed as a quotient of the sums of all the numbers of series thus increased divided by the number of years; since in the case under study the series covers 11 years, the discharge thus computed is denoted as Q_{11};
(3) using the Foster method, the frequency of the disastrous rise Q_k is computed from the increased series as a first approximation (say, the

probability of occurrence of a rise Q_k, thus obtained, amounts to 50 years;

(4) a mean annual maximum discharge Q_{10} is established from the existing series irrespective of the disastrous highwater Q_k; this discharge is denoted by Q_{10}, because in this case, it is a mean discharge of 10 observations;

(5) a mean maximum discharge Q_o for the period K ($K = 50$), in which a disastrous rise Q_k appeared is, computed from the following correlation:

$$Q_o = \frac{Q_{10}(K-1) + Q_k}{K}$$

A mean maximum discharge thus established is regarded as reliable for subsequent computations. This method of computation was supplemented in 1946, by Boldakov who recommended that it be determined in these computations as to whether the existing observations come from the wet or dry period.

This is the method of including the maximum discharge into the short period of observations.

The determination of the period can be conducted by various methods, i.e.:

(a) by analogy to the adjoining observation stations, for which values of discharges covering a longer period are known,
(b) by comparison of results of the precipitation observations,
(c) on the basis of evidence of local residents,
(d) on the basis of the thickness of annual rings in the cross section of old trees.

There are somewhat different computation methods for the observations of wet periods and dry. For the wet period, the computation should be conducted by the second method, shown above including the known disastrous discharge Q_k in a series, and another variant should be prepared without consideration of this discharge.

For a dry period, it is necessary to include the disastrous discharge Q_k in the series; but the value obtained as a first approximation might be satisfactory if the coefficient of variation is lower than 0.5.

Note that the mean discharge among those determined by various methods cannot be taken as valid. Instead, one of the discharges computed should be chosen which seems to be the most probable.

Computing Discharges with Various Probabilities Lacking Observation Data

Determining maximum rises of required probability is most difficult in river cross sections where no observations have been conducted. In such cases, indirect methods should be applied. Knowledge of the mean discharge of the

highest rises and coefficients of variation and skewness is necessary for establishing discharges with required probability of occurrence.

There are several methods of determining the mean of the highest values of the multiannual discharge, despite lack of observations.

Method 1

The value of the mean multiannual discharge is computed by the coefficient of terrain A, which may be assumed as constant for a climatically uniform terrain. For this purpose:

(a) a terrain coefficient A_o is determined, on the basis of a general formula using the known mean discharge of the highest rises Q_o of an adjoining river, situated

$$A_o = \frac{Q_o'}{F_1^n}$$

or

$$A_o = \frac{Q_o'}{F_1^n B_1^m i_1^{0.25}}$$

(b) the magnitude of the mean discharge of the highest rises Q_o is established from the correlation:

$$Q_o = A_o F^n$$

or

$$Q_o = A_o F^n B^m i^{0.25}$$

The following denotations are used in these formulas:

Q_o — mean discharge of the highest rise during a multiannual period,
F — catchment basin area,
i — catchment basin slope,
n,m — coefficients taken from Table 54,
B — catchment basin area width determined as a quotient of the catchment basin area and its length L — a line approximately dividing the catchment basin into halves.

Table 54

Value of Coefficients n and m

Type of rise	n	m
Formed by snow thawing at the following river current directions:		
a) from south to north	0.75	0.25
b) from north to south	0.67	0.25
Formed by rains	0.25	0.33

Method 2

The value of the mean multiannual discharge is determined from the correlation between the median line of an area within which appear the fluctuations of highest waters on the declivities of a valley flat, and the frequency of flooding in such area.

In this case, the mean discharge of the highest discharges is determined from the equation:

$$Q_o = a\,Q_1$$

where:

Q_1 — discharge in a main channel, the water level on the declivities of a valley flat corresponding with the mean stage between the highest and lowest maximum water stages,

a — coefficient depending on the frequence of flooding in the valley flat, determined from Table 55.

Table 55

Value of the Coefficient a

Probability of flooding the Valley Flats	a
Every year	1.4
Once in 2-3 years	1.1
Once in 4-6 years	1.0
Once in 6-10 years	0.9

Method 3

The value of the mean discharge of the highest discharges determined on the basis of traces left by highwaters on the river banks and walls of structures; highwaters cause:

(a) washing off of moss and coloring of rocks occurring under the influence of the action of sun rays,

(b) change in the color of stones in the abutment of old bridges and other structures.

The mean multiannual highwater should be computed by the several intermediate methods, outlined above, comparing the results obtained and choosing the most probable value.

The coefficients of variation and skewness can best be determined by analogy to the adjoining rivers.

A discharge with a required probability is computed using the Foster tables, after establishing Q_o, C_v and C_s.

Recalculation of Rises of known Probability into Other Rises

For recalculating rises of known probability into rises of another probability it is possible to use Table 56, as presented by Boldakov. In preparing this table, some correlations have been adopted between the coefficient of variation C_v and coefficient of skewness C_s, which may also serve for determining the coeffi-

cient of skewness when the value of the coefficient of variation is known. There are the following correlations:

C_v	C_s
0.25	$3C_v$
1.00	$2C_v$
3.00	C_v

Example:

The discharge with frequency 1 : 50 in Korzyce on the Biała River is equal to 1612 cu m/sec. A discharge of probability 1 : 100 is to be found for this cross section.

We choose a column in Table 56, for which $C_v = 0.8$, $C_s = 1.8$, because these values are nearest to the values valid for the Koszyce cross section ($C_v = 0.81$ and $C_v = 1.87$). Then:

$$Q_{100} = \frac{1612 \times 3.8}{3.3} = 1856 \text{ cu m/sec}$$

The value of the discharge with probability 1 : 100 computed for this cross section on the basis of the distributive series is equal to 1809 cu m/sec.

Table 56

Coefficients for Recalculating Rises

Probability of rises	Coefficients of variation C_v												
	0.25	0.3	0.4	0.5	0.6	0.7	0.8	1.0	1.2	1.5	2.0	2.5	3.0
	Coefficients of skewness C_s												
	0.8	0.9	1.1	1.4	1.5	1.6	1.8	2.0	2.2	2.4	2.6	2.8	3.0
1 : 5	1.2	1.2	1.3	1.4	1.4	1.5	1.5	1.6	1.7	1.8	2.0	2.1	2.2
1 : 10	1.3	1.4	1.5	1.7	1.9	2.0	2.1	2.3	2.5	2.9	3.5	4.0	4.5
1 : 25	1.5	1.6	1.8	2.0	2.3	2.5	2.8	3.2	3.7	4.3	5.5	6.7	7.8
1 : 33	1.5	1.6	1.9	2.2	2.4	2.7	2.9	3.5	4.0	4.8	6.0	7.3	8.6
1 : 50	1.6	1.7	2.0	2.3	2.6	3.0	3.3	3.9	4.5	5.5	7.0	8.6	12.0
1 : 100	1.7	1.9	2.2	2.6	3.0	3.4	3.8	4.6	5.5	6.7	8.7	11.0	13.0
1 : 300	1.9	2.1	2.5	3.0	3.5	4.0	4.5	5.6	6.7	8.5	11.0	14.0	18.0
1 : 500	2.0	2.2	2.7	3.3	3.8	4.4	5.0	6.2	7.5	9.5	13.0	16.0	20.0
1 : 1000	2.1	2.3	2.9	3.6	4.2	4.8	5.5	6.9	8.2	11.0	15.0	18.0	23.0

6. Remarks on Computing Maximum Discharges and Parameters of Distributive Series

The following principles should be applied in computing maximum discharges with a defined probability of occurrence:

(1) the distributive series of maximum annual discharges should be as long as possible, in any case, it cannot consist of a lower quantity of numbers than 10—15; individual discharges should be read from the discharge rating curve $Q = f(H)$;

(2) coefficient of variation C_v should be computed from the quantity of numbers in series higher than 25 by means of the formula:

$$C_v = \sqrt{\frac{\sum (K-1)^2}{n}}$$

the following formula should be used for the lower quantity of numbers in series:

$$C_v = \sqrt{\frac{\sum (K-1)^2}{n-1}}$$

(3) if no observation data are available for maximum discharges, the coefficient C_v may be computed for the spring rises on the basis of an analogous river, using the same formulas, or it may be determined from the approximate formulas of Polyakov, Sokolovskii and Shevelov; the coefficient of variation for summer rains can be computed — when observation data is lacking — by means of an analogous river;

(4) coefficient of skewness C_s is usually computed from the formula:

$$C_s = \frac{2C_v}{1 - K_{min}}$$

coefficient of skewness can also be adopted as follows:

(a) $C_s = 2C_v$ for spring rises,

(b) $C_s = 4C_v$ for summer rises;

(5) to check the computations:

(a) values of the annual maximum discharges observed are plotted in the scale of probability, the corresponding percentage probability being computed from the formula:

$$p = \frac{m - 0.3}{n + 0.4} 100$$

(b) the theoretical integral curve of probability is drawn, computed from the parameters Q_o, C_v and C_s;

(6) discharges with defined probability Q_p are computed by the Foster method on the basis of parameters Q_o, C_v and C_s, and applying the Foster-Rybkin table according to the formula:

$$Q_p = Q_o (fC_v + 1)$$

(7) water level corresponding with the discharge of a defined probability is established from the curve $Q = f(H)$, or the integral curve of probability of water stages drawn by the methods already described.

7. Computing Probability of Rises by means of Empirical Formulas

If no observations are available, the maximum discharges with defined probability of occurrence (frequency) can be computed by means of empirical formulas.

Some of the empirical formulas for computing maximum discharges with various probability have been derived on the basis of material concerning the intensity of heavy rains; the rest — on the basis of direct observations of maximum discharges.

The GOST 3999-48 regulations, published in the Soviet Union in 1948, do not provide for the application of the formulas of the first group; they are too approximate on account of the impossibility of practical determination of the true runoff formed by heavy downpours and on account of the adoption of many inaccurate premises in deriving formulas of that type.

When observations are lacking, these regulations recommend establishing the magnitudes of discharges on the basis of analogy to the adjoining, already investigated catchment basins, or computing them from empirical formulas derived by means of the maximum discharges observed in the field.

At the same time, the GOST regulations recommend computations by several methods and empirical formulas and accept as valid the magnitude of a maximum discharge obtained on the basis of an analysis of all the computations.

Empirical formulas for determining maximum discharges with various probability may be divided into three groups. The first of these includes formulas enabling the computation of maximum discharges formed by spring rises and the second — by summer rises.

The formulas, by means of which maximum discharges with defined probability of spring as well as summer rises can be established, belong to the third group.

Empirical Formulas for Computing Discharges of Spring Rises

Sokolovskii Formula (1937)

The Sokolovskii formulas are based on observations of maximum spring discharges in 563 river cross sections.

These formulas are not complicated and generally yield satisfactory results.

Depending on the size of catchment basin F, Sokolovskii's formulas have the following forms:

$Q = AF$ for $F < 50$ sq km
$Q = AF^{0.85}$ for F between 50 and 100 sq km
$Q = AF^{0.75}$ for $F > 100$ sq km

where:

Q — numerical value of maximum discharge in cu m/sec
A — parameter,
F — catchment basin area in sq km.

To derive the formulas, Sokolovskii established a correlation between the unit maximum runoffs and the area of catchment basin in various terrains, obtaining the following equation:

$$q = \frac{A}{F^n}$$

where:

q — value of maximum runoff in cu m/sec per sq km,
A — parameter,
n — exponent, the value of which fluctuates for particular areas within limits of 0.22 and 0.31,
F — area of catchment basin in sq km.

Sokolovskii assumed a constant value of the exponent n equal to 0.25, thus eventually arriving at the following formula for larger catchment basins:

$$q = \frac{A}{F^{0.25}}$$

or

$$Q = \frac{AF}{F^{0.25}} = AF^{0.75}$$

In computing maximum discharges in catchment basins with lakes and marshes, the following coefficient of reduction of maximum discharge should be introduced:

$$\delta = 1.0 - 0.6 \log (1 + \alpha + 0.2\,\beta)$$

where:

α — area of lakes f as a percentage of a general area of the catchment basin, i.e. $\alpha = \frac{f}{F} 100$,

β — area of marshes f' as a percentage of a general area of the catchment basin, i.e. $\beta = \frac{f'}{F} 100$.

The coefficient δ is introduced when lakes cover over 2 percent of the area of an entire catchment basin, and marshes — over 10 percent.

In afforested catchment basins, a coefficient of reduction of the maximum discharge γ should be also introduced:

$$\gamma = 1 - \lambda (p - p_1)$$

where:

p — catchment basin forest cover not as a percentage computed from the formula $p = \frac{f_z}{F}$ (f_z — area of forest cover),

p_1 — mean of afforestation of the entire area taken into account in the derivation of the parameter A,

coefficient with magnitude 0.3 for broadleaved forests in the south, and 0.6 for forests in the north.

Taking into consideration the influence of lakes, marshes and forest, Sokolovskii's formula has the following form:

$$Q = AF^{0.75}\,\delta\,\gamma$$

The Sokolovskii method is primarily based on the application of the integral parameter A, characterizing the maximum spring discharge. The values of this parameter for the territories of the USSR are shown in the map of isolines.

Determining the parameter A from the map of isolines is a very inaccurate method, because it cannot include all the individual features of catchment basins. Therefore, the method of analogy is considered fundamental for computing the value of parameter A.

When this method is applied to the computation of parameter A, Sokolovskii recommends the use of all the direct observations of maximum discharges in a given catchment basin, or in a given area, applying the formula (at $\delta = \gamma = 1$):

$$A = \frac{Q}{F^{0.75}}$$

The use of the method of analogy is admissible when reliable multiannual observations are available for catchment basins resembling as to physio-geographical character the catchment basin under study.

Since the parameter A in Sokolovskii formulas is expressed in mm/hr, coefficient $k = 0.278$ should be introduced to convert it into cu m/sec. Then, the Sokolovskii general formula will be as follows:

$$Q = 0.278\,AF^{n}$$

If the parameter A is determined by discharges appearing on an analogous river and having definite probability (e. g. 1 percent) of occurrence, the maximum discharge established by the Sokolovskii formula, using this parameter, will also correspond with 1 percent probability.

In practice, it is often easier to compute the parameter A for mean values of runoff, in this case, the methods of mathematical statistics are applied to convert discharges into other probabilities.

For this purpose, a computation is made of the value of the coefficient of variation C_v, which can be determined by means of an analogous river. Morever, the following approximate Polyakov formula serves for approximate computation of the coefficient of variation of spring maxima:

$$C_v' = 1.97\, C_v^{0.73}$$

where:

C_v' — coefficient of variation of maximum discharges,

C_v — coefficient of variation of mean annual discharges.

According to the GOST-3999-48, the following value of the coefficient of skewness is assumed for spring rises:

$$C_s = 2C_v$$

After determining the values of coefficients, presented above the maximum discharges with various frequencies are established by means of the Foster table.

Example

The length of a river from its sources to the cross section under study, amounts to L — 72 km, and the area of its catchment basin — 650 sq km. The river flows across lowland terrain, where lakes cover 1 percent of the entire catchment basin, marshes — 12 percent, and forests — 32 percent. The valley flat of the river is narrow and, therefore, the accumulation of maximum discharges is quite insignificant.

Compute the maximum spring discharge with probability 1 : 50 by means of the Sokolovskii formula:

$$Q = AF^{0.75}\delta\gamma$$

Since maximum spring discharges are known in this cross section for a 5-year period only (highest during this period — 92 cu m/sec and mean — 64 cu m/sec) they cannot serve for computing discharge with a probability of 1 : 50 and, therefore, such discharge should be established on the basis of a selected analogous river. For this purpose, the physical and geographical conditions of adjoining rivers should be examined and one of them should be selected as the analogue.

The area of catchment basin of a selected analogous river from its sources to the cross section assumed amounts to 980 sq km, length — 81 km. Lakes cover 0.5 percent of the catchment basin area, marshes — 28 percent, and forests — 35 percent.

The hydrological observations were conducted in the cross section of the analogous river selected for 22 years, the highest spring discharge amounting during this interval to 180 cu m/sec. and the mean spring discharge was 92 cu m/sec.

$C_v = 0.38$ and $C_s = 0.82$ are determined for the analogous river by means of a distributive series of maximum discharges and, subsequently, the discharge Q_{50} (with frequency 1 : 50) is computed using the Foster tables:

$$Q_{50} = Q_o\,(f\,C_v + 1) = 92\,(2.54 \times 0.38 + 1) = 181 \text{ cu m/sec}$$

Then, the value of parameter A_{50} is determined for the analogous river by the formula:

$$A_{50} = \frac{Q_{50}}{F^{0.75}} = \frac{181}{980^{0.75}} = \frac{181}{175} = 1.04 \text{ cu m/sec.}$$

It is required to ascertain whether the influence of forests, lakes and marshes is similar in both these catchment basins.

The quantity (percentage) of forests in both catchment basins is almost identical and, therefore, no corrections of discharges are necessary in this respect.

The area of lakes is below 2 percent of the entire area of each of these basins and, therefore, lakes have practically no influence on the decrease in maximum discharges.

But the quantity of marshes in both catchment basins differs considerably ($\beta = 12$ percent in the cross section under study, and $\beta = 28$ percent in the catchment basin of the analogous river). Consequently, their influence on the decrease in maximum discharges will be different, too.

To establish the influence of marshes, the value of coefficients of reduction of maximum discharges δ and δ' should be computed for each catchment basin; they will amount to:

$$\delta = 1 - 0.6 \, log\,(1 + 0.2\,\beta) = 1 - 0.6\, log\,(1 + 0.2 \times 12) = 0.68$$

for the river under study and:

$$\delta' = 1 - 0.6\, log\,(1 + 0.2 \times 28) = 0.51$$

for the analogous river.

Then, the ratio k of these coefficient is established:

$$k = \frac{\delta}{\delta'} = \frac{0.68}{0.51} = 1.33$$

After substituting the values thus obtained, the volume of the maximum discharge with probability 1 : 50 is determined for the river cross section under study, from the formula:

$$Q_{50} = \frac{AF^{0.75}\,\delta}{\delta'} = AF^{0.75}\,k = 1.04 \times 650^{0.75} \times 1.33 =$$

$$= 1.04 \times 129 \times 1.33 = 178 \text{ cu m/sec}$$

Computing Discharges by the Method of Boundary Intensities of Snow Melting

This method is based on the observation that during the period of snow melting in small catchment basins, there occur rises every day, reaching their peak at midday and subsiding in the evening or at night. Consequently the general quantity of snow is of no major importance, the intensity of snow thawing and possibility of rains being the chief factors. For this reason, the volume of the spring discharges in small catchment basins located near Cracow will be greater than in, for instance, the vicinities of Białystok — although the general reserve of water in the snow in the latter area is usually greater than in the Cracow region.

It may be assumed that an intensive snow thawing begins at 10 A. M. and ends at 8 P. M., while the peak intensity of snow melting is usually reached at 2 P. M.

This method can be used in catchment basins where maximum volumes of discharge of the snow thawing, together with rains, do not exceed a magnitude of 1,000 cu m/sec.

The discharge per second is computed, when using this method, from the formula:

$$Q = \frac{W \cdot F \cdot n}{0.5 \times 10 \times 60 \times 60} \times \frac{4}{4 + \tau} = \frac{W \cdot F \cdot n}{4500\,(4 + \tau)}$$

where:

Q — discharge in cu m/sec,

W — volume of water in cu m running off one sq km (Table 57),

Table 57

Water Volume W in Thousands of cu m per one sq km

Probability $1:l$	W	Probability $1:l$	W
1 : 2	7	1 : 100	29
1 : 5	11	1 : 300	36
1 : 10	15	1 : 1000	43
1 : 25	21	1 : 10000	52
1 : 50	25	1 : 1000000	60

F — area of catchment basin in sq km,

n — coefficient of the characteristics of the water runoff equal to 1.0 for catchment basins F larger than 100 sq km, and for catchment basins smaller than 100 sq km computed from the formula $n = 2.5 - 0.15\,F$ (Table 58)

τ — duration in hours of the peak flood in a rise after 2 P. M., depending on the time required for the formation of maximum runoffs on decliv-

ities and in a bed; for the latter computation the formula $\tau = 1.5\tau_l L_o$ is used, where L_o — distance of the center of gravity of the catchment basin area from the structure in hectometers, and τ_l time of water runoff in a bed in minutes (Table 60) depending on the bed slope and on the morphological coefficient of the river bed surface, resistance m_1 exerting its influence on the water velocity in a channel (Table 59).

The duration of an intensive water runoff from beginning to end (in hours) is determined by the following formula:

$$\frac{10\,(4+\tau)}{4} = 10 + 2.5\,\tau$$

Table 58

Value of Coefficient of the Runoff Characteristics n

F sq km	n	F sq km	n
1	2.4	30	1.7
2	2.3	40	1.5
3	2.2	50	1.4
5	2.15	60	1.3
7	2.1	70	1.2
10	2.0	80	1.2
15	1.9	90	1.1
20	1.8	100	1.0

Example

The area of a catchment basin $F = 40$ sq km, the slope of an overgrown bed $i = 5$ per mil, distance of the center of gravity of the catchment basin from the structure $L_o = 5$ km $= 50$ hectometers.

Table 59

Value of Coefficient of the River Bed Surface Resistance m_1

Type of surface of the river bed	m_1
Smooth earth bed	25
Meandering or overgrown bed	20
Highly overgrown bed strewn with stones	15
Bed consisting of boulders and coarse stones	10

To be computed: the volume of the spring rise with frequency 1 : 25.

$W = 21{,}000$ cu m is assumed from Table 57 for frequency 1 : 25, and the value of the coefficient $n = 1.5$ is read from Table 58 for a catchment basin

Table 60

Duration τl in Minutes of Water Runoff along the River Bed for every 100 m. of River Bed Length

Q in cu m/sec	m_1	Mean slope of river bed in per mil											
		1	2	3	5	7	10	15	20	30	40	60	100
3	25	4.6	3.5	2.9	2.5	2.2	1.9	1.7	1.5	1.3	1.1	0.97	0.85
	20	5.5	4.1	3.6	2.9	2.6	2.3	1.9	1.8	1.5	1.3	1.20	0.95
	15	6.6	5.1	4.5	3.6	3.2	2.8	2.4	2.2	1.8	1.7	1.50	1.20
	10	—	6.9	6.0	4.9	4.3	3.8	3.3	3.0	2.5	2.3	2.0	1.5
5	25	4.1	3.1	2.6	2.2	1.9	1.7	1.5	1.3	1.1	1.0	0.85	0.75
	20	4.8	3.6	3.2	2.6	2.3	2.0	1.7	1.6	1.3	1.2	1.0	0.80
	15	5.9	4.5	4.0	3.2	2.8	2.5	2.2	1.9	1.6	1.5	1.3	1.0
	10	—	6.1	5.2	4.3	3.8	3.4	2.9	2.6	2.2	2.0	1.6	1.4
7	25	3.7	2.8	2.4	2.0	1.8	1.5	1.4	1.2	1.0	0.9	0.78	0.70
	20	4.4	3.3	2.9	2.4	2.1	1.8	1.6	1.4	1.2	1.1	0.92	0.75
	15	5.4	4.1	3.9	2.9	2.6	2.3	2.0	1.8	1.5	1.4	1.2	0.94
	10	—	5.6	4.8	3.9	3.5	3.1	2.7	2.4	2.0	1.8	1.5	1.3
10	25	3.4	2.6	2.2	1.8	1.7	1.4	1.3	1.1	0.93	0.84	0.75	0.63
	20	4.0	3.0	2.6	2.2	1.9	1.6	1.4	1.3	1.1	0.97	0.85	0.71
	15	5.0	3.7	3.3	2.7	2.4	2.1	1.8	1.6	1.4	1.2	1.1	0.86
	10	—	5.1	4.4	3.7	3.2	2.9	2.4	2.2	1.8	1.7	1.4	1.2
15	25	3.0	2.3	2.0	1.6	1.5	1.3	1.2	1.0	0.9	0.75	0.65	0.55
	20	3.6	2.8	2.4	1.9	1.7	1.5	1.3	1.2	1.1	0.9	0.77	0.60
	15	4.5	3.4	2.9	2.4	2.1	1.9	1.6	1.5	1.3	1.1	0.95	0.76
	10	—	4.6	3.9	3.2	2.9	2.6	2.2	1.9	1.7	1.5	1.3	1.2
20	25	2.6	2.1	1.9	1.5	1.4	1.2	1.1	0.93	0.79	0.71	0.60	0.50
	20	3.4	2.6	2.2	1.8	1.6	1.4	1.2	1.1	0.94	0.84	0.72	0.58
	15	4.2	3.2	2.7	2.2	2.0	1.7	1.5	1.4	1.2	1.0	0.88	0.74
	10	5.6	4.3	3.6	3.1	2.7	2.4	2.0	1.8	1.6	1.4	1.2	1.0
50	25	2.2	1.7	1.5	1.2	1.1	0.9	0.85	0.73	0.60	0.55	0.46	0.40
	20	2.7	2.1	1.8	1.4	1.3	1.1	0.95	0.86	0.75	0.65	0.56	0.45
	15	3.3	2.5	2.2	1.8	1.6	1.4	1.2	1.1	0.9	0.82	0.66	0.55
	10	4.4	3.4	2.8	2.5	2.2	1.9	1.6	1.4	1.3	1.1	0.96	0.82
100	25	1.9	1.4	1.2	1.0	0.9	0.8	0.7	0.6	0.50	0.45	0.40	0.34
	20	2.3	1.8	1.5	1.2	1.1	0.94	0.8	0.72	0.62	0.55	0.46	0.40
	15	2.7	2.1	1.8	1.5	1.3	1.1	1.0	0.9	0.78	0.70	0.57	0.50
	10	3.7	2.8	2.4	2.1	1.8	1.6	1.4	1.2	1.10	0.95	0.82	0.67

Q in cu m/sec	m_1	Mean slope of river bed in per mil											
		1	2	3	5	7	10	15	20	30	40	60	100
200	25	1.6	1.2	1.1	0.86	0.77	0.67	0.58	0.52	0.44	0.40	0.35	0.29
	20	1.9	1.4	1.3	1.0	0.91	0.79	0.68	0.61	0.52	0.47	0.40	0.34
	15	2.3	1.8	1.6	1.3	1.1	0.97	0.84	0.76	0.64	0.58	0.50	0.41
	10	3.2	2.4	2.1	1.7	1.5	1.3	1.2	1.0	0.86	0.79	0.68	0.56
500	25	1.1	0.96	0.85	0.69	0.62	0.54	0.46	0.41	0.36	0.31	0.28	0.23
	20	1.2	1.1	1.0	0.81	0.73	0.65	0.56	0.49	0.43	0.38	0.33	0.26
	15	1.7	1.4	1.3	1.0	0.88	0.78	0.69	0.60	0.53	0.46	0.40	0.33
	10	2.3	2.0	1.4	1.4	1.2	1.1	0.92	0.82	0.70	0.62	0.54	0.45
1000	25	0.96	0.81	0.72	0.59	0.52	0.45	0.40	0.35	0.31	0.28	0.23	0.19
	20	1.1	0.98	0.85	0.68	0.61	0.53	0.46	0.41	0.36	0.32	0.27	0.22
	15	1.4	1.2	1.0	0.86	0.76	0.65	0.59	0.51	0.44	0.38	0.33	0.28
	10	1.9	1.6	1.4	1.1	0.97	0.88	0.77	0.69	0.59	0.53	0.45	0.38

area equal to 40 sq km. The duration of the passing of the peak wave of the rise after 2 P. M. is computed from the formula:

$$'\tau = 1.5\,\tau_l\,L_o$$

duration of water runoff in a river bed τ_l appearing in this formula is determined from Table 60. For this purpose, an arbitrary discharge — e. g., 50 cu m/sec — is assumed and then, for the river bed slope equal to 5 per mil and for m_1 equal to 20 will amount to 1.4 and $\tau = 1.5_{\tau l}\,L_o = 1.5 \times 1.4 \times 50 = 105$ minutes $=$ $= 1.75$ hours.

After substituting these values into the formula for computing maximum discharges, the following values will be arrived at:

$$Q = \frac{WFn}{4500\,(4+\tau)} = \frac{21000 \times 40 \times 1.5}{4500\,(4+1.75)} = 48.5 \text{ cu m/sec}$$

The discharge computed at 48.5 cu m/sec differs by only 3 percent from the assumed discharge (50 cu m/sec) and may therefore, be accepted as reliable. Should the volume of the discharge obtained from the formula differ from the assumed volume by more than 5 percent, the computation should be repeated assuming as a starting point the discharge obtained that is 48.5.

The number of hours from the beginning to the end of an intensive runoff is determined from the formula:

$$10 + 2.5\,\tau = 10 + 2.5 \times 1.75 = 14.4 \text{ hours}$$

i. e., the end of the intensive runoff will take place after midnight.

Empirical Formulas for Computing Discharges Produced by Summer Rains

Many empirical formulas have been derived for computing maximum discharges with various probabilities caused by summer rains. Some of them facilitate the computation of discharges in small catchment basins; others are suitable for application to larger catchment basins.

Sokolovskii Formula

In 1945 Sokolovskii presented the following formula for computing maximum discharges Q cu m/sec caused by summer rains:

$$Q = \frac{0.28\, H\, \alpha\, F}{t}\, f\, \delta' \lambda + Q'$$

where:

H — quantity in mm of precipitation in T hours,

α — coefficient of flow of the peak wave of a rise with required frequency, which is assumed as being equal to the coefficient of flow of the entire volume of a rise,

F — catchment basin area in sq km,

t — duration of the water rise in hours equal to the duration of the water runoff in a stream bed,

f — coefficient of the shape of a wave during a rise caused by rain,

δ' — coefficient depending on the number of lakes and marshes located in a catchment basin,

λ — coefficient in respect of the regulating influence exerted by the shape of a river channel,

Q' — additional discharge formed by ground water fed to the river, in cu m/sec.

The following formula is used computing the reliable quantity of precipitation H:

$$H = S\,(60\,T)^{1/3}$$

where:

T — duration of rain in hrs,

S — variable precipitation parameter in mm/min.

An opinion has been expressed by Professor Ogievski that since Sokolovskii accepted a very generalized value of the coefficient of flow α the introduction into this formula of a variable value of parameter S, fluctuating within close limits, is quite unjustified. The acceptance of a constant value of S would cause a much diminished error than that resulting from the adoption of very approximate values of coefficent of flow α.

On the basis of this opinion, it is reasonable to suggest, that the constant mean values of parameter S shown in Table 61 should be used for Polish territories.

Duration of rain T in hrs is computed from the formula:

$$T = \mu t = t(t+1)^{-0.2}$$

Coefficient μ is called by Sokolovskii the coefficient of flow delay. Values μ and $T = \mu t$ depending on time t are presented in Table 62.

Table 61

Values of the Precipitation Parameter S in mm/min

Probability	1 : 10	1 : 25	1 : 50	1 : 100	1 : 200
Value of parameter S in mm/min	8.25	9.85	10.05	12.25	13.45

Table 62

Values of Coefficient μ and Rain Duration T

Duration of water rise t in hours	μ	Duration of precipitation in hours $T = \mu t$
1	0.95÷0.90	1.0
2	0.80	1.6
10	0.62	6.2
24	0.52	12.5
36	0.49	18.0
48	0.46	22.0
72	0.42	30.0
120	0.38	45.0

Sokolovskii proves that the duration of water rise t in small catchment basins can be assumed as equal to the duration of water runoff in a river channel, i.e.:

$$t = \frac{L}{3.6\,v} \text{ hours}$$

where:

L — river length in km from the sources to the cross section under study,

v — maximum mean velocity in the cross section in m/sec.

Velocity v can be computed from the Chezy formula, or from the following correlation presented by Sokolovskii and Pokrovskii:

$$v = 17\, i^{0.4} h^{0.5}$$

where:

v — mean velocity in m/sec,

i — longitudinal slope of the water level expressed by a decimal fraction,
h — mean depth in m.

The Sokolovskii-Pokrovskii formula can be applied on rivers with well developed channels, with slope $i < 0.10$. This formula may give too high velocities of water current in small, poorly developed channels overgrown with vegatation.

The approximate mean velocities in a cross section can be determined according to Table 63.

Table 63

Approximate Mean Velocities v m/sec during Rises

Catchment basin configuration	Small rivers with depths smaller than 1 m during floods	Average and large rivers
Lowland	$0.3 \div 0.5$	$0.4 \div 0.8$
Plains	$0.8 \div 1.2$	$1.0 \div 1.5$
Hilly or foothills	$1.5 \div 2.5$	$2.0 \div 2.5$
Mountains	$2.5 \div 3.5$	$2.5 \div 4.0$

Coefficient of flow α is determined from the formula:

$$\alpha = \frac{0.28\,HFfQ_w}{t}$$

where: Q_w — maximum discharge in cu m/sec established by traces left in the cross section by the highwater.

Other denotations — identical as for the former formula.

Approximate values of coefficient α are presented in Table 64.

Table 64

Coefficients of Flow α of the Flood

Catchment basin configuration	Probability of discharges in %						
	0.3	1	2	3	5	10	50
Lowland	0.25	0.20	0.15	0.12	0.09	0.07	0.04
Plains	0.35	0.30	0.25	0.20	0.17	0.15	0.10
Foothill	0.40	0.35	0.30	0.25	0.22	0.20	0.12
Mountains	0.45	0.40	0.35	0.30	0.27	0.25	0.15

Note: For catchment basins with permeable ground the values of coefficient can be reduced by 30—50 percent

Coefficient of the shape of a graph of a flood wave f depends on the curves of the water rise and fall. Its value amounts to:

$$f = \frac{12}{4 + 3\gamma}$$

Small rivers have $\gamma = 2.0$ and, therefore, their $f = 1.2$, average and large rivers without any significant valley flats have $\gamma = 2.5 - 3.0$ and, therefore, their $f = 1.04 - 0.92$, large rivers with wide valley flats have $\gamma = 4.0$ and $f = 0.75$.

If lakes or marshes are located in the catchment basin, the reduction of maximum discharges due to the accumulation of rain water in lakes and marshes should be taken into account. The accumulation has a higher influence on the summer than on spring rises because of the smaller volume of summer rises as compared with spring rises.

For this reason, coefficient δ' is somewhat different for summer than for spring rises, and namely:

$$\delta' = 1.0 - 0.7 \log (1 + a_o + 0.2 \beta_o)$$

where:

α_o = percentage of ratio of lake areas to the area of the entire catchment basin,

β_o — percentage of ratio of marsh areas to the area of the entire catchment basin.

The magnitude of the reduction coefficients for summer and spring rises with an identical number of lakes and marshes are shown in Table 65.

Table 65

Values of Coefficients for Summer (δ') and Spring (δ) Rises

Percentage of lakes and marshes	Values of coefficients	
	δ	δ'
3	0.70	0.62
5	0.58	0.44
10	0.40	0.20
20	0.22	0.00

Table 65 shows that the reduction of summer maxima is higher than the spring maxima and the difference between corresponding reduction coefficients for summer and spring rises increases together with an increase in the area of lakes and marshes.

The additional volume of discharge Q' arising as a result of ground water

being fed to the river, is approximately equal to the multiannual discharge in a river. The discharge Q' is taken into account in large and medium catchment basins, but may be neglected in small ones. The Sokolovskii formula may be used only for approximate calculation of summer discharge of varied probability since results in some cases may be rather inaccurate.

Example

To be computed: the maximum discharge, with probability once in a 100 years, on a stream of length from the sources to the cross section under study $L = 8$ km, and catchment basin area $F = 22$ sq km. The catchment basin of the stream has a lowland character; there are no lakes or marshes and the groundwater feeding is very low.

In view of the existing conditions — $\delta' = 1$, $\mu = 1$ and $Q' = 0$, — the Sokolovskii formula acquires the following form:

$$Q = \frac{0.28\, H \alpha F f}{t}$$

The water velocity $v = 1$ m/sec is taken from Table 63 and, therefore, the duration of the water rise in hours will amount to:

$$t = \frac{L}{3.6\, v} = \frac{8}{3.6 \times 1} = 2.22 \text{ hours}$$

Coefficient of the flow delay:

$$\mu = (t + 1)^{-0.2} = 3.22^{-0.2} = 0.80$$

Duration of rain in hours:

$$T = \mu t = 0.8 \times 2.22 = 1.8 \text{ hour}$$

Valid quantity of precipitation (H) will amount to:

$$H = S\,(60\, T)^{1/3} = 12.25\,(60 \times 1.8)^{1/3} = 58 \text{ mm}$$

Coefficient of flow α is taken from Table 64; for frequency 1 : 100 coefficient $\alpha = 0.30$. For small rivers, $\gamma = 2$, then we obtain:

$$f = \frac{12}{4 + 3\gamma} = \frac{12}{4 + 3 \times 2} = 1.2$$

The maximum discharge of summer rains with frequency 1 : 100 will amount to:

$$Q = \frac{0.28\, H \alpha F f}{t} = \frac{0.28 \times 58 \times 0.30 \times 22 \times 1.2}{2.22} = 58.3 \text{ cu m/sec}$$

Abbrieviated DORNII Formula for Computing Maximum Summer Discharges with Various Probability

To determine discharges with various probability the following formula can be used, prepared by the Soviet Scientific Road Research Institute (DORNII):

$$Q = \frac{n}{t} F^{1/2} h^{3/2}$$

where:

Q — volume of discharge with assumed probability in cu m/sec,
n — coefficient, of value $n = 3$ for lowland catchment basins, $n = 3.5$ for hilly catchment basins, and $n = 4.0$ for mountain catchment basins,
t — duration of runoff in minutes, taken usually to be within limits of 20 and 45 mins,
h — layer of flow in mm — selected from Tables 66, 67 and 68.

The magnitude of flow h depends on:

(1) catchment basin elevation above sea level,
(2) assumed probability of rise,
(3) category of soil,
(4) retentive capacity of the soil Z taken from Table 14.

The magnitudes of flow are presented in Tables 66, 67 and 68 for 6 categories of soil (or catchment basin surface), namely:

category I — impermeable rocky surface without cracks, asphalt, concrete,
„ II — clay,
„ III — rich black earth, sandy-clayey ground,
„ IV — ordinary black earth, sandy-clayey ground covered with turf,
„ V — sandy-clayey ground without turf,
„ VI — sand.

The thicknesses of the flow layer in mm with ground retentive capacity $Z = 0$, are presented in Tables 66, 67 and 68. If the retentive capacity of terrain Z is greater than zero, the value Z is subtracted from the thickness of the layer of flow h shown in Tables 66, 67 and 68.

Example

To be computed: the discharge of summer rains of probability 1 : 100, in the cross section of a stream with a catchment basin area F — 64 sq km. The

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Table 66

Thickness of the Flow Layer $h$mm for Lowland Rivers (Except Rivers of the Marine Region) at $Z = 0$

Duration of runoff t min	Probability of discharge $1:l$						
	5	10	25	50	100	300	1000
	Ground category I						
20	12	17	21	28	34	41	53
30	16	24	27	34	42	50	63
45	21	30	33	41	49	57	70
60	25	35	39	48	55	64	78
80	26	36	41	50	60	68	85
100	28	38	42	52	62	72	88
120	33	44	50	60	70	80	100
150	35	47	53	63	73	87	105
200	38	50	57	70	79	92	113
300	43	53	63	77	87	101	121
	Ground category II						
20	8	13	17	25	30	36	48
30	11	18	22	29	35	43	55
45	14	23	26	35	41	49	62
60	16	25	29	37	45	53	68
80	17	28	31	40	48	57	75
100	17	28	32	43	51	60	80
120	18	29	33	43	53	64	84
150	18	29	34	45	55	65	87
200	18	29	35	47	58	68	90
300	15	27	35	47	60	70	95
	Ground category III						
20	—	10	14	21	26	34	46
30	—	13	17	24	30	39	52
45	—	16	21	27	35	43	56
60	—	17	23	30	38	46	60
80	—	17	24	32	41	49	66
100	—	17	24	33	42	52	70
120	—	16	25	33	43	54	74
150	—	15	24	34	43	55	75
200	—	11	22	33	42	55	76
300	—	3	15	29	38	54	75

Duration of runoff t min	Probability of discharge 1 : l						
	5	10	25	50	100	300	1000
	Ground category IV						
20	—	—	11	16	22	30	42
30	—	—	13	20	27	36	48
45	—	—	15	22	30	38	54
60	—	—	15	23	31	39	56
80	—	—	14	22	32	40	58
100	—	—	7	21	31	40	60
120	—	—	—	19	30	40	60
150	—	—	—	17	29	40	60
200	—	—	—	14	27	38	59
300	—	—	—	6	22	38	56
	Ground category V						
20	—	—	—	—	15	20	33
30	—	—	—	—	13	20	32
45	—	—	—	—	9	17	32
60	—	—	—	—	—	15	31
80	—	—	—	—	—	10	31
100	—	—	—	—	—	4	29
120	—	—	—	—	—	—	27
150	—	—	—	—	—	—	23
200	—	—	—	—	—	—	11
300	—	—	—	—	—	—	—
	Ground category VI						
20	—	—	—	—	—	8	21
30	—	—	—	—	—	5	18
45	—	—	—	—	—	—	8
60	—	—	—	—	—	—	—
80	—	—	—	—	—	—	—
100	—	—	—	—	—	—	—
120	—	—	—	—	—	—	—
150	—	—	—	—	—	—	—
200	—	—	—	—	—	—	—
300	—	—	—	—	—	—	—

Thickness of flow Layer h mm for the Foothill Region and Mountain Rivers at $Z = 0$

Duration of runoff t min	Probability of discharge 1 : l						
	5	10	25	50	100	300	1000
	Ground category I						
20	18	20	30	33	44	69	80
30	22	26	37	40	51	75	90
45	25	29	41	46	56	80	100
60	28	32	44	51	61	85	105
80	32	36	47	56	67	91	112
100	34	38	50	62	72	95	117
120	35	40	53	66	76	98	120
150	37	45	59	71	82	103	126
200	43	53	66	80	90	112	137
300	55	66	80	93	105	128	156
	Ground category II						
20	14	18	21	29	41	65	83
30	16	20	23	33	44	68	86
45	17	22	27	36	48	73	91
60	19	24	30	40	53	75	95
80	20	25	33	44	56	78	99
100	20	26	35	48	59	81	100
120	21	27	38	51	62	83	105
150	23	30	40	54	67	85	110
200	25	33	45	60	74	92	120
300	28	40	55	69	83	101	133
	Ground category III						
20	9	12	17	25	36	58	73
30	11	14	19	28	39	61	78
45	11	15	21	30	42	65	85
60	11	15	23	33	45	68	90
80	11	15	25	35	46	71	94
100	11	16	25	38	48	73	95
120	11	17	27	40	50	73	96
150	12	19	29	42	52	74	97
200	13	21	32	44	55	78	102
300	15	25	35	48	60	84	110

Duration of runoff t min	Probability of discharge $1:l$						
	5	10	25	50	100	300	1000
	Ground category IV						
20	—	7	17	26	35	56	75
30	—	7	18	28	37	59	77
45	—	7	19	28	39	61	80
60	—	7	19	28	39	62	82
80	—	8	19	28	39	62	83
100	—	9	19	28	39	62	83
120	—	9	20	29	40	62	84
150	—	10	21	30	41	63	85
200	—	12	23	31	44	65	90
300	—	15	27	35	48	68	96
	Ground category V						
20	—	—	—	5	25	46	63
30	—	—	—	5	22	44	63
45	—	—	—	5	20	42	63
60	—	—	—	5	18	36	62
80	—	—	—	5	15	31	58
100	—	—	—	5	15	30	55
120	—	—	—	5	15	30	54
150	—	—	—	3	15	30	54
200	—	—	—	—	15	30	45
300	—	—	—	—	15	30	45
	Ground category VI						
20	—	—	—	—	—	18	43
30	—	—	—	—	—	15	40
45	—	—	—	—	—	10	32
60	—	—	—	—	—	—	30
80	—	—	—	—	—	—	25
100	—	—	—	—	—	—	25
120	—	—	—	—	—	—	25
150	—	—	—	—	—	—	25
200	—	—	—	—	—	—	25
300	—	—	—	—	—	—	25

Thickness of flow Layer h mm. for Rivers of the Marine Region between the Rivers Vistula and Oder at $Z = 0$

Duration of runoff t min	Probability of discharge $1:l$						
	5	10	25	50	100	300	1000
	Ground category I						
20	18	20	24	27	32	37	40
30	21	25	29	32	38	43	46
45	24	30	34	38	44	50	56
60	26	32	38	42	49	55	64
80	29	36	42	48	54	61	73
100	31	39	45	52	59	67	79
120	33	41	49	55	62	71	84
150	35	44	52	60	67	77	91
200	38	47	55	64	72	83	100
300	44	55	62	71	79	90	109
	Ground category II						
20	13	16	19	22	28	33	35
30	15	20	23	26	30	37	40
45	17	22	27	30	35	41	48
60	18	24	29	33	39	46	55
80	18	25	32	36	42	50	60
100	18	25	33	38	45	53	64
120	18	26	34	40	47	55	68
150	18	28	35	42	50	59	72
200	18	28	35	43	50	62	78
300	14	25	33	44	53	63	82
	Ground category III						
20	10	12	17	19	24	29	33
30	11	15	19	22	27	33	37
45	11	16	20	24	30	36	42
60	10	16	22	25	32	38	46.
80	8	15	23	27	33	40	50
100	6	14	23	28	34	42	54
120	5	14	23	28	35	44	57
150	—	12	22	28	35	46	60
200	—	10	18	26	35	47	64
300	—	—	8	17	28	41	63

Duration of runoff t min	Probability of discharge $1:l$						
	5	10	25	50	100	300	1000
	Ground category IV						
20	8	9	13	16	22	26	28
30	7	10	15	18	23	29	32
45	—	10	15	19	24	30	37
60	—	—	14	19	24	30	40
80	—	—	—	19	24	31	43
100	—	—	—	17	24	31	44
120	—	—	—	14	22	31	45
150	—	—	—	—	18	30	45
200	—	—	—	—	10	37	45
300	—	—	—	—	—	18	40
	Ground category V						
20	—	—	—	9	15	16	18
30	—	—	—	—	3	15	18
45	—	—	—	—	—	6	19
60	—	—	—	—	—	—	18
80	—	—	—	—	—	—	16
100	—	—	—	—	—	—	13
120	—	—	—	—	—	—	8
150	—	—	—	—	—	—	—
200	—	—	—	—	—	—	—
300	—	—	—	—	—	—	—

90 percent flat catchment basin consists of meadows grown with sparse grass and plowed soil. The soil, partially sandy covered with turf and partially black earth, belongs to category IV.

We apply the formula:

$$Q_{100}=\frac{n}{t}F^{1/2}h^{3/2}$$

The coefficient of the terrain retentive capacity $Z=10$ mm is selected from Table 14 for meadows with sparse grass and black earth. The coefficient $n=3$ is assumed for the flat terrain. Table 66 valid for flat areas is used for computing the thickness of flow layer h. Various durations of water runoff $t=20, 30, 45$ min are assumed.

For $t = 20$ min, and category IV soil, $h = 22 - 10 = 12$ mm; for $t = 30$ min, $h = 27 - 10 = 17$ mm, for $t = 45$ min, $h = 30 - 10 = 20$ mm.

The maximum discharges Q_{100} are subsequently computed by means of the results thus obtained.

For $t = 20$ min and $h = 12$ mm:

$$Q_{100} = \frac{n}{t} F^{1/2} h^{3/2} = \frac{3}{20} \times 64^{1/2} \times 12^{3/2} = 50 \text{ cu m/sec}$$

For $t = 30$ min and $h = 17$ mm:

$$Q_{100} = \frac{3}{30} \times 64^{1/2}\, 17^{3/2} = 56 \text{ cu m/sec}$$

For $t = 45$ min and $h = 20$ mm:

$$Q_{100} = \frac{3}{45} \times 64^{1/2} \times 20^{3/2} = 48 \text{ cu m/sec}$$

The results thus obtained indicate that the highest discharge $Q = 56$ cu m/sec, which we take as valid in our computation for a bridge span will occur at an outflow with a layer thickness $h = 17$ mm during a period of 30 minutes.

8. Polish Formulas for Computing Discharges with Various Probabilities

Formulas by which maximum discharges with various probabilities can be computed were recently elaborated in Poland. The Dębski and Jarocki formulas belong to this group.

Dębski Formulas

The following formula was presented by Dębski for computing the discharge of the median water among the highest discharges Q_o i. e., with a probability of occurrence together with higher stages amounting to 50 percent:

$$Q_o = CA^{2/3}$$

where:

Q_o — median discharge of the highest discharges in cu m/sec,
C — coefficient of the characteristics of efflux selected from Table 69,
A — catchment basin area in sq km.

The following formula is recomended by Dębski for computing maximum discharges with various frequencies:

$$Q_{p\%} = Q_o\,[1 + C_v f(s)]$$

where:

$Q_{p\%}$ — maximum discharge in cu m/sec, with probability of occurrence p percent,

Table 69

Value of Coefficients C for the Dębski Formula for Computing Discharge Q_o

No.	River	Locality	Catchment basin A sq km	Coefficient C
1	Oder	Chałupki	4596	1.828
2		Olza	5839	1.680
3		Racibórz	6698	1.612
4		Chrapkowice	10760	1.301
5		Kopin	18035	0.937
6		Brzeg	19723	0.877
7		Brzeg Dolny	26461	0.672
8		Małoszyn	26848	0.672
9		Ścinawa	29605	0.615
10		Chobień	30338	0.616
11		Cigacice	39913	0.593
12		Nietkowice	40549	0.593
13		Połęsko	47293	0.597
14		Przybrzeg	52033	0.598
15		Słubice	53580	0.599
16		Kiniec	109093	0.613
17		Gozdowice	109364	0.613
18		Zatoń Górna	109564	0.613
19		Słowicze Piaski	112143	0.614
20	Warta	Mrzygłód	68.4	0.516
21		Korwinów	549.5	0.554
22		Bobry	1822.1	0.576
23		Działoszyn	4100.7	0.592
24		Burzenin	5442.4	0.598
25		Sieradz	8185.0	0.606
26		Uniejów	9189.5	0.608
27		Koło	12040.8	0.614
28		Gorzów	51893.0	0.374
29		Świerkocin	52278.0	0.374
30	Widawka	Podgórze	2377.0	0.327
31	Grabia	Łask	471.3	0.310
32	Ner	Chocianowice	120.0	0.296
33	Prosna	Podzamcze	1233.7	0.392
34		Piwonice	2931.6	0.403
35		Bogusław	4352.0	0.408
36		Ruda Komorska	4862.8	0.410

Table 69 (continued)

No.	River	Locality	Catchment basin A sq km	Coefficient C
37	Vistula	Jawiszowice	909.5	1.847
38		Pustynia	3848.0	1.304
39		Dwory	5240.0	2.088
40		Cracow	8021.0	2.118
41		Karsy	19784.0	2.124
42		Sandomierz	33358.0	2.160
43		Annopol	51605.0	1.963
44		Puławy	57303.0	1.970
45		Warsaw	85176.0	1.996
46		Płock	168362.0	1.336
47		Tczew	193170.0	1.342
48	Czarna Przemsza	Piwoń	165.8	0.430
49	Biała Przemsza	Sławków	476.0	0.396
50		Maczki	608.0	0.400
51	Przemsza	Jeleń	1957.0	0.461
52		Chełmek	2045.0	0.462
53	Soła	Czernichów	1032.0	4.077
54		Oświęcim	1388.0	4.118
55	Skawa	Jordanów	(60.0)	1.867
56		Osielec	(203.0)	1.944
57		Wadowice	838.0	2.038
58	Białucha	Dąbie	(190.0)	0.428
59	Raba	Gdów	(953.0)	2.047
60	Czarny Dunajec	Nowy Targ	440.0	3.342
61	Dunajec	Waksmund	700.2	3.394
62		Nowy Sącz	4345.0	2.339
63		Tropie	4890.0	2.349
64		Siedliszowice	6813.0	2.375
65	Nida	Brzegi	2204.0	1.440
66	Czarna Nida	Tokarnia	1193.0	1.375
67	Wisłoka	Zółków	587.0	2.276
68	Wisłoka	Korzeniów	3477.0	2.415
69	San	Przemyśl	3675.8	2.186
70		Jarosław	7035.8	2.234
71		Radomyśl	16749.9	2.300
72	Solinka	Terka	307.6	2.013
73	Osława	Zagórz	500.5	2.046
74	Wieprz	Zwierzyniec	(346.5)	0.390

No.	River	Locality	Catchment basin A sq km	Coefficient C
75		Krasnystaw	2999.1	0.419
76		Łęczna	(4255.0)	0.424
77		Kośmin	10573.0	0.436
78	Radomka	Ryczywół	2120.0	0.358
79	Pilica	Szczekociny	353.6	0.518
80		Tomaszów	4953.9	0.565
81		Nowe Miasto	6688.1	0.571
82		Warka	8987.4	0.576
83	Żebrówka	Bonowice	127.0	0.394
84	Bug	Brześć	22496.2	0.556
85		Tonkiele	30638.1	0.562
86		Małkinia	33853.3	0.563
87		Wyszków	38159.3	0.566
88		Zegrze	67764.0	0.577
89	Krzna	Neple (Malowa Góra)	2938.8	0.456
90	Nurzec	Brańsk	1182.0	0.443
91		Ciechanowiec	1806.0	0.449
92	Kostrzyń	Jagodne	532.8	0.431
93	Narew	Strękowa Góra	6980.0	0.581
94		Wizna	14066.0	0.594
95		Nowogród	15398.0	0.596
96		Ostrołęka	21276.0	0.603
97		Rożan	23764.0	0.605
98		Pułtusk	27705.0	0.608
99		Wierzbica	28320.0	0.608
100	Orzyc	Chorzele	810.0	0.431
101		Krasnosielec	1225.0	0.227
102		Maków	1950.0	0.444
103	Wkra	Cieksyn	5033.0	0.458
104	Bzura	Łowicz	3490.0	0.338
105	Sucha	Wola Miedniewska	103.9	0.301
106	Stara Rzeka	Wola Miedniewska	47.1	0.294
107	Sucha	Sucha Nowa	177.5	0.307
108	Drwęca	Nowe Miasto	2775.0	0.831
109		Brodnica	3693.0	0.839
110		Nowa Wieś	5388.6	0.850
111	Brda	Ciecholewy	682.0	0.993

Q_o — median discharge of the highest discharges in cu m/sec,
C_v — coefficient of variation,
$f(s)$ — function of the coefficient of asymmetry (skewness).

The computation of maximum discharges with various probabilities by means of the Dębski formula is primarily based on the following three specific values of the distributive series:

(1) discharge Q_1, which is less than 10 percent of other discharges in a distributive series: the place where Q_1 appears in a distributive series is called by Dębski the upper decile;
(2) median discharge Q_o less 50 percent of other discharges;
(3) discharge Q_2, less than 90 percent of other discharges; this place in a distributive series is called the lower decile.

The successive numbers of the upper decile m_1, median value m_o and lower decile m_2 in a distributive series are determined by the following formulas:

$$m_1 = \frac{n}{10} + 0.5$$

$$m_o = \frac{n}{2} + 0.5$$

$$m_2 = \frac{9n}{10} + 0.5$$

where n — quantity of numbers in a series.

The successive numbers of maximum discharges m_1, m_o and m_2 being determined in a distributive series, the values of discharges Q_1, Q_o and Q_2 are read directly from the distributive series of maximum discharges or from the averaged integral curve of frequency drawn in the scale of probability in places where $p = 10$ percent, 50 percent and 90 percent.

Coefficients of variation C_v and skewness s are computed from the formulas:

$$C_v = \frac{Q_1 - Q_2}{2Q_o}$$

$$s = \frac{2(Q_1 + Q_2 - 2Q_o)}{Q_1 - Q_2}$$

Tables 70 and 71, where the ordinates of functions of asymmetry $f(s)$ are shown for positive and negative coefficients, are used in subsequent computations.

Example

Compute by the Dębski formula the maximum discharge with probability of occurrence 1 : 100 on the River Biała in Koszyce, using the series of observations over a 43-year period.

Probability p percent	Coefficients of skewness $s > 0$										
	0.0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
0.1	2.41	2.58	2.74	2.91	3.08	3.25	3.42	3.58	3.75	3.92	4.08
0.2	2.24	2.38	2.53	2.67	2.82	2.96	3.10	3.25	3.39	3.53	3.67
0.4	2.07	2.19	2.31	2.43	2.55	2.67	2.79	2.91	3.03	3.15	3.27
0.5	2.01	2.12	2.23	2.35	2.46	2.57	2.69	2.80	2.91	3.03	3.14
1.0	1.81	1.90	1.99	2.08	2.17	2.26	2.35	2.44	2.53	2.62	2.72
2.0	1.60	1.67	1.74	1.81	1.88	1.95	2.02	2.09	2.16	2.23	2.30
2.5	1.53	1.59	1.66	1.72	1.79	1.85	1.91	1.97	2.03	2.09	2.16
4.0	1.36	1.41	1.46	1.51	1.56	1.61	1.66	1.71	1.76	1.81	1.85
5.0	1.28	1.32	1.37	1.41	1.46	1.50	1.54	1.59	1.63	1.67	1.71
10.0	1.00	1.02	1.05	1.07	1.10	1.12	1.15	1.17	1.20	1.22	1.25
20.0	0.66	0.67	0.68	0.69	0.70	0.71	0.72	0.73	0.74	0.75	0.76
25.0	0.52	0.52	0.53	0.54	0.54	0.55	0.56	0.56	0.57	0.57	0.58
30.0	0.41	0.42	0.42	0.42	0.43	0.43	0.43	0.44	0.44	0.44	0.45
40.0	0.20	0.20	0.20	0.20	0.20	0.20	0.21	0.21	0.21	0.21	0.21
50.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
60.0	−0.20	−0.20	−0.20	−0.20	−0.20	−0.19	−0.19	−0.19	−0.19	−0.19	−0.19
70.0	−0.41	−0.41	−0.40	−0.40	−0.39	−0.39	−0.38	−0.38	−0.37	−0.37	−0.36
75.0	−0.52	−0.51	−0.51	−0.50	−0.49	−0.48	−0.48	−0.47	−0.46	−0.46	−0.45
80.0	−0.66	−0.65	−0.64	−0.63	−0.62	−0.60	−0.59	−0.58	−0.57	−0.56	−0.55
90.0	−1.00	−0.97	−0.95	−0.92	−0.90	−0.87	−0.85	−0.83	−0.80	−0.78	−0.75
95.0	−1.28	−1.24	−1.20	−1.16	−1.12	−1.08	−1.04	−1.00	−0.96	−0.92	−0.88
96.0	−1.36	−1.31	−1.27	−1.22	−1.18	−1.13	−1.09	−1.04	−1.00	−0.66	−0.91
97.5	−1.53	−1.47	−1.41	−1.36	−1.30	−1.25	−1.19	−1.14	−1.09	−1.02	−0.99
98.0	−1.60	−1.53	−1.47	−1.41	−1.35	−1.29	−1.23	−1.17	−1.11	−1.05	−1.01
99.0	−1.81	−1.73	−1.65	−1.57	−1.49	−1.42	−1.34	−1.27	−1.20	−1.13	−1.08
99.5	−2.01	−1.91	−1.81	−1.71	−1.62	−1.53	−1.44	−1.36	−1.28	−1.21	−1.13

for Positive Coefficients of Skewness (Asymmetry)

Coefficients of skewness $s > 0$													
0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.05	1.10	1.15	1.20
4.25	4.41	4.58	4.75	4.92	5.09	5.25	5.45	5.59	5.76	5.93	6.09	6.26	6.43
3.81	3.96	4.10	4.25	4.39	4.54	4.68	4.83	4.97	5.11	5.25	5.40	5.54	5.69
3.39	3.51	3.63	3.75	3.87	3.99	4.11	4.23	4.23	4.35	4.59	4.71	4.83	4.95
3.25	3.36	3.48	3.59	3.70	3.82	3.93	4.04	4.16	4.27	4.38	4.49	4.61	4.72
2.81	2.90	2.99	3.08	3.17	3.26	3.35	3.44	3.53	3.63	3.72	3.81	3.90	3.99
2.37	2.44	2.51	2.58	2.65	2.72	2.79	2.86	2.93	2.99	3.06	3.13	3.20	3.27
2.22	2.29	2.35	2.41	2.48	2.54	2.60	2.66	2.72	2.79	2.85	2.91	2.97	3.04
1.90	1.95	2.00	2.05	2.10	2.15	2.20	2.25	2.30	2.34	2.39	2.44	2.49	2.54
1.75	1.80	1.84	1.89	1.93	1.97	2.02	2.06	2.10	2.14	2.18	2.23	2.27	2.32
1.27	1.30	1.32	1.35	1.37	1.40	1.42	1.45	1.47	1.50	1.52	1.55	1.57	1.60
0.77	0.78	0.79	0.80	0.81	0.82	0.83	0.84	0.85	0.86	0.87	0.88	0.89	0.90
0.59	0.60	0.61	0.61	0.62	0.63	0.63	0.64	0.64	0.65	0.66	0.66	0.67	0.67
0.45	0.45	0.46	0.46	0.46	0.47	0.47	0.47	0.48	0.48	0.49	0.49	0.49	0.50
0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.22	0.22	0.22	0.22
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
−0.19	−0.19	−0.19	−0.18	−0.18	−0.18	−0.18	−0.18	−0.18	−0.18	−0.17	−0.17	−0.16	−0.16
−0.36	−0.35	−0.35	−0.34	−0.34	−0.33	−0.32	−0.32	−0.32	−0.31	−0.29	−0.29	−0.28	−0.27
−0.44	−0.43	−0.42	−0.41	−0.41	−0.40	−0.38	−0.38	−0.37	−0.35	−0.34	−0.33	−0.31	−0.30
−0.54	−0.52	−0.51	−0.50	−0.49	−0.47	−0.45	−0.45	−0.44	−0.41	−0.40	−0.38	−0.36	−0.35
−0.73	−0.70	−0.68	−0.65	−0.63	−0.60	−0.57	−0.55	−0.53	−0.50	−0.47	−0.45	−0.42	−0.40
−0.85	−0.81	−0.78	−0.74	−0.71	−0.67	−0.64	−0.61	−0.58	−0.55	−0.51	−0.48	−0.45	−0.42
−0.88	−0.84	−0.80	−0.76	−0.73	−0.69	−0.65	−0.62	−0.59	−0.55	−0.52	−0.49	−0.46	−0.43
−0.94	−0.90	−0.85	−0.81	−0.77	−0.73	−0.68	−0.65	−0.62	−0.58	−0.54	−0.51	−0.47	−0.44
−0.96	−0.91	−0.87	−0.82	−0.78	−0.73	−0.69	−0.66	−0.62	−0.58	−0.54	−0.51	−0.47	−0.44
−1.02	−0.97	−0.91	−0.86	−0.82	−0.77	−0.72	−0.68	−0.64	−0.60	−0.56	−0.52	−0.49	−0.45
−1.07	−1.01	−0.95	−0.89	−0.85	−0.79	−0.74	−0.70	−0.66	−0.62	−0.58	−0.54	−0.50	−0.46

Probability p per-cent	Coefficients of skewness $s < 0$											
	0.0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55
0.1	2.41	2.26	2.12	1.99	1.86	1.73	1.62	1.51	1.41	1.32	1.23	1.15
0.2	2.24	2.11	1.99	1.87	1.76	1.64	1.54	1.45	1.36	1.27	1.19	1.12
0.4	2.07	1.96	1.86	1.75	1.66	1.56	1.47	1.38	1.30	1.23	1.15	1.09
0.5	2.01	1.91	1.81	1.71	1.62	1.53	1.44	1.36	1.28	1.21	1.13	1.07
1.0	1.81	1.73	1.65	1.57	1.49	1.42	1.34	1.27	1.20	1.13	1.08	1.02
2.0	1.60	1.53	1.47	1.41	1.35	1.29	1.23	1.17	1.11	1.05	1.01	0.96
2.5	1.53	1.47	1.41	1.36	1.30	1.25	1.19	1.14	1.09	1.02	0.99	0.94
4.0	1.36	1.31	1.27	1.22	1.18	1.13	1.09	1.04	1.00	0.96	0.91	0.88
5.0	1.28	1.24	1.20	1.16	1.12	1.08	1.04	1.00	0.96	0.92	0.88	0.85
10.0	1.00	0.97	0.95	0.92	0.90	0.87	0.85	0.83	0.80	0.78	0.75	0.73
20.0	0.66	0.65	0.64	0.63	0.62	0.60	0.59	0.58	0.57	0.56	0.55	0.54
25.0	0.52	0.51	0.51	0.50	0.49	0.48	0.48	0.47	0.46	0.46	0.45	0.44
30.0	0.41	0.41	0.40	0.40	0.39	0.39	0.38	0.38	0.37	0.37	0.36	0.36
40.0	0.20	0.20	0.20	0.20	0.20	0.19	0.19	0.19	0.19	0.19	0.19	0.19
50.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
60.0	−0.20	−0.20	−0.20	−0.20	−0.20	−0.20	−0.21	−0.21	−0.21	−0.21	−0.21	−0.21
70.0	−0.41	−0.42	−0.42	−0.42	−0.43	−0.43	−0.43	−0.44	−0.44	−0.44	−0.45	−0.45
75.0	−0.52	−0.52	−0.53	−0.54	−0.54	−0.55	−0.56	−0.56	−0.57	−0.57	−0.58	−0.59
80.0	−0.66	−0.67	−0.68	−0.69	−0.70	−0.71	−0.72	−0.73	−0.74	−0.75	−0.76	−0.77
90.0	−1.00	−1.02	−1.05	−1.07	−1.10	−1.12	−1.15	−1.17	−1.20	−1.22	−1.25	−1.27
95.0	−1.28	−1.32	−1.37	−1.41	−1.46	−1.50	−1.54	−1.59	−1.63	−1.67	−1.71	−1.75
96.0	−1.36	−1.41	−1.46	−1.51	−1.56	−1.61	−1.66	−1.71	−1.76	−1.81	−1.85	−1.90
97.0	−1.53	−1.59	−1.66	−1.72	−1.79	−1.85	−1.91	−1.97	−2.03	−2.09	−2.16	−2.22
98.0	−1.60	−1.67	−1.74	−1.81	−1.88	−1.95	−2.02	−2.09	−2.16	−2.23	−2.30	−2.37
99.0	−1.81	−1.90	−1.99	−2.08	−2.17	−2.26	−2.35	−2.44	−2.53	−2.62	−2.72	−2.81
99.5	−2.01	−2.12	−2.23	−2.35	−2.46	−2.57	−2.69	−2.80	−2.91	−3.03	−3.14	−3.25

Table 71

Negative Coefficients of Skewness (Asymmetry).

Coefficients of skewness $s < 0$												
0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.05	1.10	1.15	1.20
1.08	1.01	0.95	0.89	0.83	0.77	0.73	0.69	0.64	0.59	0.55	0.51	0.47
1.05	0.99	0.92	0.87	0.81	0.76	0.72	0.68	0.63	0.58	0.54	0.50	0.47
1.03	0.97	0.90	0.86	0.80	0.75	0.71	0.67	0.62	0.58	0.54	0.50	0.47
1.01	0.95	0.89	0.85	0.79	0.74	0.70	0.66	0.62	0.58	0.54	0.50	0.46
0.97	0.91	0.86	0.82	0.77	0.72	0.68	0.64	0.60	0.56	0.52	0.49	0.45
0.91	0.87	0.82	0.78	0.73	0.69	0.66	0.62	0.58	0.54	0.51	0.48	0.44
0.90	0.85	0.81	0.77	0.73	0.68	0.65	0.62	0.58	0.54	0.51	0.47	0.44
0.84	0.80	0.76	0.73	0.69	0.65	0.62	0.59	0.55	0.52	0.49	0.46	0.43
0.81	0.78	0.74	0.71	0.67	0.64	0.61	0.58	0.55	0.51	0.48	0.45	0.42
0.70	0.68	0.65	0.63	0.60	0.57	0.55	0.53	0.50	0.47	0.45	0.42	0.40
0.52	0.51	0.50	0.49	0.47	0.45	0.45	0.44	0.41	0.40	0.38	0.36	0.35
0.43	0.42	0.41	0.41	0.40	0.38	0.38	0.37	0.35	0.34	0.33	0.31	0.30
0.35	0.35	0.34	0.34	0.33	0.32	0.32	0.32	0.31	0.29	0.29	0.28	0.27
0.19	0.19	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.17	0.17	0.16	0.16
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
−0.21	−0.21	−0.21	−0.21	−0.21	−0.21	−0.21	−0.21	−0.21	−0.22	−0.22	−0.22	−0.22
−0.45	−0.46	−0.46	−0.46	−0.47	−0.47	−0.47	−0.48	−0.48	−0.49	−0.49	−0.49	−0.50
−0.60	−0.61	−0.61	−0.62	−0.63	−0.63	−0.64	−0.64	−0.65	−0.66	−0.66	−0.67	−0.67
−0.78	−0.79	−0.80	−0.81	−0.82	−0.83	−0.84	−0.85	−0.86	−0.87	−0.88	−0.89	−0.90
−1.30	−1.32	−1.35	−1.37	−1.40	−1.42	−1.45	−1.47	−1.50	−1.52	−1.55	−1.57	−1.60
−1.80	−1.84	−1.89	−1.93	−1.97	−2.02	−2.06	−2.10	−2.14	−2.18	−2.23	−2.27	−2.32
−1.95	−2.00	−2.05	−2.10	−2.15	−2.20	−2.25	−2.30	−2.34	−2.39	−2.44	−2.49	−2.54
−2.29	−2.35	−2.41	−2.48	−2.54	−2.60	−2.66	−2.72	−2.79	−2.85	−2.91	−2.97	−3.04
−2.44	−2.51	−2.58	−2.65	−2.72	−2.79	−2.86	−2.93	−2.99	−3.06	−3.13	−3.20	−3.27
−2.90	−2.99	−3.08	−3.17	−3.26	−3.35	−3.44	−3.53	−3.63	−3.72	−3.81	−3.90	−3.99
−3.36	−3.48	−3.59	−3.70	−3.82	−3.93	−4.04	−4.16	−4.27	−4.38	−4.49	−4.61	−4.72

The successive number of the upper decile amounts to:

$$m_1 = \frac{n}{10} + 0.5 = \frac{43}{10} + 0.5 = 4.8$$

The discharge volume 1,103 cu m/sec corresponds with the fourth number of the distributive series and 856 cu m/sec with the fifth number. The interpolation for $m_1 = 4.8$ yields the discharge $Q = 1{,}103 - 0.8 \times 247 = 1{,}103 - 197.6 = 905.4$ cu m/sec — i. e. rounded up, 905 cu m/sec.

The successive number of the median value of the series is equal to:

$$m_o = \frac{n}{2} + 0.5 = 22$$

$Q_o = 391$ cu m/sec corresponds with this number.

The successive number of the lower decile has the following value:

$$m_2 = \frac{9n}{10} + 0.5 = \frac{9 \times 43}{10} + 0.5 = 39.2$$

The discharge amounting to 139 cu m/sec corresponds with the number 39 of the series, and 126 cu m/sec with number 40. The discharge in the place $m_2 = 39.2$ computed by extrapolation, amounts to: $Q_2 = 139 - 0.2 \times 13 = 136$ cu m/sec.

The coefficients of variation C_v and skewness s are subsequently computed by means of the following equations:

$$C_v = \frac{Q_1 - Q_2}{2Q_o} = \frac{905 - 136}{2 \times 391} = \frac{769}{782} = 0.98$$

$$s = \frac{2(Q_1 + Q_2 - 2Q_o)}{Q_1 - Q_2} = \frac{2(905 + 136 - 782)}{769} = \frac{518}{769} = 0.67$$

The discharge Q_{100} is computed by means of Table 70 and on the basis of the following formula:

$$Q_{100} = Q_o [1 + C_v f(s)] = 391 (1 + 0.98 \times 3.03) = 391 \times 3.97 = 1552 \text{ cu m/sec.}$$

Jarocki Formula

To compute discharges with various probability of occurrence, the use of the following empirical formula is suggested by the present author:

$$Q_p = Q_o + (Q_{max} - Q_o)\, a_p$$

where:

Q_p — discharge with probability $1 : p$ years,

Q_o — mean discharge of the highest discharges during the period of observations,

Q_{max} — maxiumm discharge during this period,
a_p — coefficient of probability selected from Table 72.

For the number of years of observation or for the probability, which are not shown in Table 72, the coefficient a_p is determined by linear interpolation.

Table 72

Values of Coefficients of Probability a_p

Number of years of observations, or periods in which the highest discharge is known	Coefficients a_p for probability									
	1 : 10	1 : 20	1 : 25	1 : 33	1 : 50	1 : 100	1 : 200	1 : 300	1 : 600	1:1000
10	1.00	1.48	1.65	1.84	2.10	2.57	(3.00)*	(3.30)	(3.78)	(4.07)
20	0.67	1.00	1.14	1.27	1.47	1.81	2.12	2.32	2.65	2.85
25	0.60	0.91	1.00	1.14	1.32	1.62	1.91	2.06	2.35	2.54
33	0.54	0.82	0.89	1.00	1.15	1.40	1.65	1.79	2.05	2.22
50	0.48	0.70	0.78	0.87	1.00	1.22	1.44	1.56	1.78	1.94
100	(0.38)	0.57	0.64	0.71	0.82	1.00	1.18	1.27	1.45	1.57
200	(0.32)	0.48	0.53	0.60	0.70	0.85	1.00	1.08	1.23	1.33

*) Computing discharges by coefficients in parentheses may yield less accurate values.

If a high discharge (one or more of such), the value of which deviates to a considerable extent from other observations, occurs during the period of observation, such a discharge should not be taken into account in computations because it is not characteristic for the appropriate period. Discharges of this type can be checked by comparing them with the highest discharge for the same year in the adjoining cross sections where the probability of occurrence is known.

If no characteristic discharges of this type are taken into account in computations, the values of discharges with various probabilities obtained by the formula suggested are greater than the true values.

If several-year intervals should not be supplemented by fictitious numbers the real observations only should be taken into account and the mean value Q_o should be computed by means of these observations only.

Only the value of the mean of the highest discharges and the maximum discharge during the period of observations are taken into account in the formula suggested. To determine these discharges, it is not necessary, as required when other formulas of this type are applied, to arrange observations in decreasing or increasing order.

Considerable devations may occur in computing probable discharges by various methods using distributive series if observations are taken not from an

average period but from either a dry or wet one; consequently, the period of observations should be as long as possible.

Example 1

The mean of the highest discharges during the 25-year period $Q_o = 480$ cu m/sec, and the highest discharge valid for this period $Q_{max} = 1{,}240$ cu m/sec. Compute the discharge with probability once in a 100 years by means of the Jarocki formula.

The coefficient of probability a_p amounting for probability 1 : 100 to $a_{100} = 1.62$ is read from Table 72 for the 25-year period of observations. The following formula is used for computing the discharge:

$$Q_p = Q_o + (Q_{max} - Q_o)\, a_p$$

After numerical values have been substituted, the following result is arrived at:

$$Q_{100} = Q_o + (Q_{25} - Q_o)\, a_{100} = 480 + (1240 - 480)\, 1.62 = 480 + 1231.2 = \\ = 1711.2 \text{ cu m/sec}$$

Example 2

The mean discharge on the River Biała in Koszyce, computed on the basis of a 43-year period of observations of maximum discharges, amounts to 470.5 cu m/sec, and the highest discharge recorded during that period is equal to 1,635 cu m/sec. Compute by means of the formula last given the discharge with probability 1 : 100.

The value of coefficient a_p for probability once in a 100 years, with observations of 43 years available, is determined by interpolation:

$$a_{100} = 1.40 - \frac{1.40 - 1.22}{50 - 33}(43 - 33) = 1.40 - 0.11 = 1.29$$

The discharge with probability 1 : 100 will, therefore amount to:

$$Q_{100} = Q_o + (Q_{43} - Q_o)\, a_{100} = 470.5 + (1635 - 470.5)\, 1.29 = \\ = 470.5 + 1502.3 = 1972.8 \text{ cu m/sec}$$

The discharge with probability 1 : 100 computed by the Foster method on the basis of the 43-year period of observations amounts to 1,809 cu m/sec. (Table 47).

9. Guiding Rules for Computing Maximum Discharges

Methods of computing maximum discharges with various probabilities are usually different for large catchment basins and small basins because observations of discharges are not generally conducted for small basins.

The following guiding rules should be applied to compute maximum discharges:

(1) if observations for at least a 10-year period are taken into account, the maximum discharges of various frequency should be computed by the Dębski formula or the Jarocki formula; the Foster method or, in the event of a large number of observations, the graphic method may be used for checking the results obtained;

(2) in the case of a complete absence of observations in catchment basins, where the discharge volume does not exceed 1,000 cu m/sec, the Soviet Scientific Road Research Institute formula should be used for computing discharges resulting from summer rains, and for discharges formed by spring rises — the method of extremal intensities of snow thawing; from the discharge computed by these formulas the higher one should be adopted as valid;

(3) in the event of a complete absence of observations or of their being insufficient in number in larger catchment basins, maximum discharges with various probabilities may be determined by the Sokolovskii formulas;

(4) when there is a complete absence of observations, maximum discharges may be established by an adequately chosen analogous river; for instance, the Sokolovskii formula for computing discharges formed by spring rises is based on the use of observations conducted in the cross section of an analogous river.

The physico-geographical conditions of an analogous river must be similar to those of the river under study. The following elements should be examined in detail, to establish this similarity:

(a) water stage observations,

(b) contour maps of catchment basins,

(c) maps of snow cover distribution,

(d) dates of appearance of spring rises,

(e) data on forests, number of lakes and marshes,

(f) the character of the valley flat in order to give a possibility of investigating the flattening of the flood wave in the event of a considerable extension of a catchment basin — because a decrease in the discharge volume then takes place.

When choosing an analogous river, it should be borne in mind, that such a river must flow from the same declivity from which the river under study flows. The conditions as regards the quantity of forests, lakes and marshes should be similar for both catchment basins. If, however, differences appear in this respect, reductive coefficients should be introduced into the computations.

CHAPTER VI

LOCATION OF BRIDGES AND CULVERTS

1. Passage across the River

Structures such as bridges or culverts, underwater tunnels, filtrating embankments and ferries serve for crossing rivers, canals and other artificial or natural water areas.

A bridge or culvert is a structure facilitating the passage over a water obstacle, while an underwater tunnel enables passage under a water obstacle (Fig. 77).

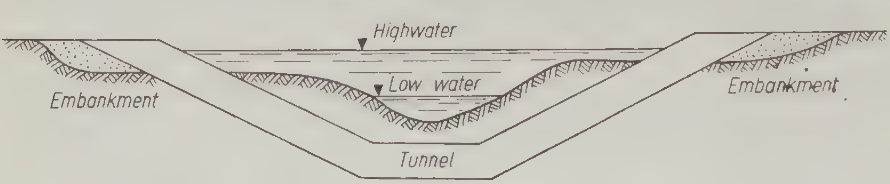

Fig. 77. Diagram of an underwater tunnel

A filtrating embankment is a road sector built of stones which passes the stream water through the openings between the stones (Fig. 78). A ferry is a mobile device for transporting people, goods and vehicles across a water obstacle (Fig. 79).

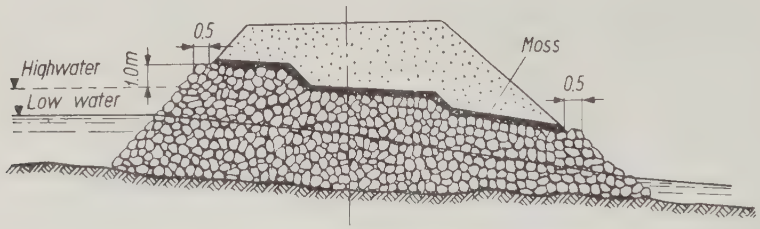

Fig. 78. Infiltration embankment

The selected type of structure serving for passage, across a water obstacle should ensure a safe and convenient crossing at minimum outlay on construction and maintenance. However, not all the structures referred to above fully meet these requirements. Underwater tunnels for instance, are very expensive, fil-

trating embankments are not in all cases suitable and secure, while ferries considerably diminish the traffic capacity of a water route. The latter two types of structures, therefore, are only temporarily used while filtrating embankments are built mostly on small streams and dried up channels, and ferries are mostly used on large rivers.

General Technical Requirements

Streams are mostly crossed by bridges and culverts.

It is best to build only one bridge (Fig. 80) in a given river cross section; several bridges (Fig. 81) should be designed in such a cross section only when local conditions permit.

A bridge may be built across the entire valley of a stream, the entire valley flat or only a part of it. In large and narrow valleys, particularly in mountain terrain, where a stream occupies little space, a bridge span larger than that indicated by computations, is sometimes adopted since considerations of building costs and traffic safety make it more advantageous to add bridge spans than to build high embankments.

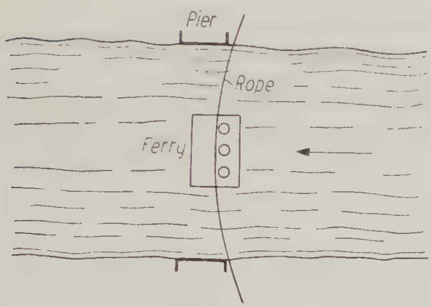

Fig. 79. Diagram of a ferry

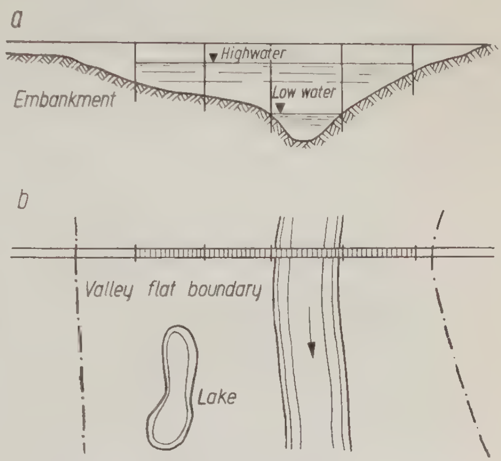

Fig. 80. Diagram of a crossing of a stream channel by a single bridge: *a* — cross section, *b* — upstream view

When designing structures on small rivers, it is desirable to consider whether, under the specific conditions, it is better to build a small bridge or a culvert.

With large and average embankments, it is more advantageous to build culverts if discharges are not very great while bridges should be constructed on streams prone to intensive ice drifts.

Culverts have many advantages by comparison with bridges, since they facilitate the laying of a uniform road surface — advantageous from the point of view of vehicular traffic — on the approaches and over the culvert. The main-

tenance of the road sector within the bridge crossing does not, in this case, differ from the maintenance of the rest of a road.

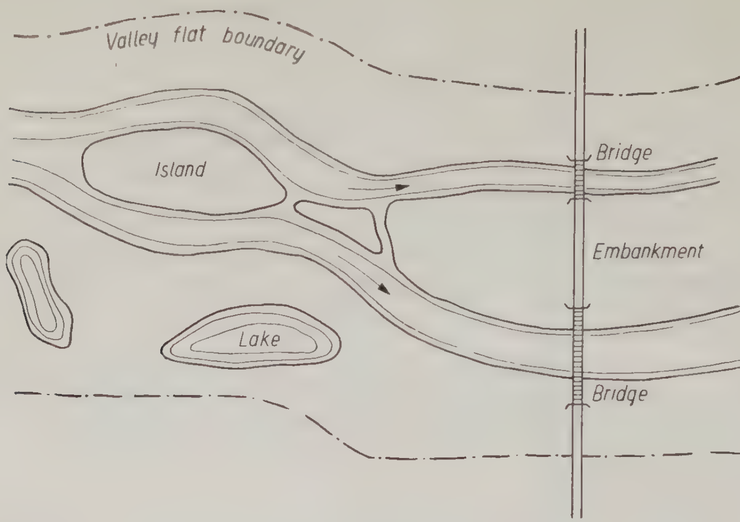

Fig. 81. Scheme of a crossing of a stream channel by two bridges

A culvert has no pillars such as constitute the principal constructional parts of a bridge, the building of which is difficult and expensive.

A decision to build culverts of a uniform type and size on a road facilitates building work and enables the use of precast elements and the introduction of mechanization onto the building site.

The flooding of the opening of a reinforced concrete culvert can be permitted; such pressure openings will be able to pass several times greater discharges. In such cases, however, a water rise should be carefully examined, lest it should cause harmful scour of the embankment, involving costly reinforcement.

The crossing of a stream is not usually limited to the building of a bridge only; several auxiliary structures should also be put up. The full set of structures serving to pass a road over the stream, and a stream under a road is called a bridge passage, and consists of:

(a) a bridge itself with its carrying construction and pillars,
(b) two-sided bridge approaches,
(c) directional dikes serving for the smooth direction of water into the bridge opening and out of the opening,
(d) realignment structures in a stream securing the stability of the channel and movement of water within the limits of the bridge passage, and in the nearest stream sector.

Not all the structures mentioned above, of course are always built in bridge

passages. The directional dikes, for instance, are not necessary if a bridge is located in a river sector having reinforced banks or dikes.

River realignment is necessary only in the event of unfavorable conditions of discharge, when, for instance, the valley flat is considerably narrowed by the bridge abutments.

It is most often, however, after the bridge is built that the necessity to re-align the stream appears; then, it shows that the bridge passage was incorrectly designed, resulting in a deterioration in the navigating conditions.

General Economic Requirements

In the selection of a bridge passage, both technical and economic aspects should be taken into account.

The passage may be considered economically favorable when the cost of building and maintaining it are at minimum and the time required for building it the shortest possible. The fulfillment of these conditions depends very often on the possibility of using cheap building materials obtainable on or near the building site.

A bridge passage should simultaneously ensure the security and continuation of traffic, a requirement which may raise the cost of building. In this case too, however, the bridge passage is considered to be economically designed if it achieves, for instance, the shortening of a road, greater possibilities of increasing traffic in the future.

Several variants of stream crossing should, therefore, be elaborated and examined from the technical and economical aspects.

The following factors should be taken into consideration in comparing individual variants:

(a) building cost of the bridge passage,
(b) cost of transportation during the first years of the exploitation period,
(c) annual cost of maintaining the passage.

In exeptional cases, the necessity may arise to choose a more expensive variant if, for instance, it is required by the navigation conditions or other important reasons.

Comparing variants from the economic point of view the future possibilities of developing the bridge passage should also be taken into account.

2. Selecting the Location of a Stream Crossing

A correctly designed bridge passage should, for minimum outlay:

(a) facilitate convenient and safe road traffic without any disturbances in the sector of the bridge passage;

(b) secure the passing of the discharge of water and ice through the opening with the least disturbance of the regimen of the stream;

(c) let vessels and rafts pass conveniently and safely under the bridge on navigable rivers and those used for log-rafting.

These conditions can be fulfilled only when the location of a stream crossing is correctly selected — usually a difficult task under various local conditions.

Conditions Decisive for the Location of a Stream Crossing

Primary efforts should be made to observe the following conditions in choosing the site for a future bridge crossing:

(1) the valley flats should be crossed at their narrowest point and the main channel at its widest;

(2) the selected sector of the stream should facilitate the designing of a route perpendicular both to the main channel and the valley of the stream; the spans of a bridge should be so distributed as to allow the main stream to pass through the center opening of the main span;

(3) large skew bridges can be built only in exceptional cases, since the piers are exposed to the impact of ice, which may cause the formation of ice barriers;

(4) a stream sector should be selected where there are no side branches, so that only one bridge need be built;

(5) in view of the possibility of sediment accumulation, a bridge should not be located directly below the mouth of a tributary;

(6) the regimen of the stream sector selected should not be subject to great changes during a year; the regimen is the most homogeneous in straight and deep sectors;

(7) if possible, the stream sector selected should have no branchings, lakes, bayous, etc., because they create favorable conditions for channel line deviations at highwater stages;

(8) the situation of the stream channel and the configuration of its bed should be stable, because changes tend to form islands and shoals; this condition is best met by locating the crossing below the center of the curvature of the stream and in the places where a bend in the stream passes into an opposite bend;

(9) the valley flat should be situated as high as possible above the low water stage, because low directional dikes are then sufficient and in these valley flats the depth is the smallest;

(10) bridge passages should be located in a river sector having stable geological structure suitable for founding bridge pillars, the constructional cost of which is high — some 50 percent of the total cost of the bridge; the size of the bridge opening also depends on the geological structure,

since a greater velocity is admissible under a bridge when the soil is more resistant to scour;

(11) a bridge passage on a navigable river and those used for rafting should facilitate convenient and safe passing of vessels and rafts through the appropriate bridge spans;

(12) if on a river in its natural state, there is no place fulfilling the requirements indicated above, the necessary conditions may be achieved by artificial means consisting in building realignment structures, reinforcing river banks, etc.

Direction of Bisecting the Stream

Efforts should be made to locate the route of a road at right angles both to the main channel and to the river valley; such a direction is more favorable for the water discharge and avoids building a skew bridge or changing the direction of the stream.

The situation of a bridge askew to the direction of the water movement exerts an unfavorable influence on the work of a structure. The scour of the river bed then occurs near the bridge pillars and the individual bridge spans work unevenly when water discharge passes through them.

If, due to the bridge approaches or for other reasons, it is impossible to locate the bridge at right angles to the main channel and the valley of the stream, it is desirable that the divergence from the perpendicular to the direction of the main channel should form an angle not greater than 5-10°. Sometimes, the perpendicular direction of a bridge can be maintained only in that part of a valley which passes about 90 percent of the discharge.

In small catchment basins, it is advisable not to curve the route, particularly on the more important roads. In such cases, it is the direction of a stream which should be changed by building an artificial river channel at right angles to the axis of a road route; the cost of such an operation is in small catchment basins not excessive (Fig. 82).

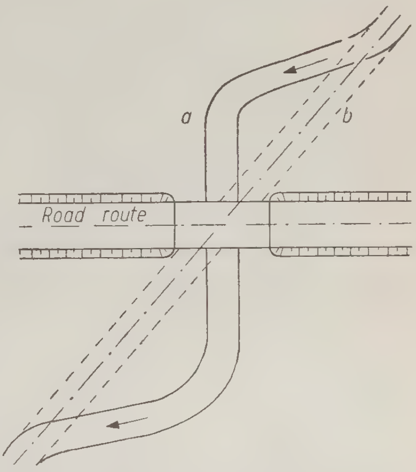

Fig. 82. Scheme of a bridge passageway: *a* — artificial stream section, *b* — natural stream section

Such a change in the direction of the stream involves shortening a culvert, but also deteriorates the conditions of water movement above and below the route in places where the channel of the stream bends. For this reason, it is more

rational to situate skew culverts with small openings, thus avoiding a change in the direction of the stream.

The necessity to place the structure aslant may also arise in the event of the appearance of rocky ground in deeply cut river valleys, since to change the direction of the stream is then expensive.

It is impossible to change the direction of a stream which conveys with it considerable quantities of sediment, because such sediment will be deposited in the vicinity of the bends of the artificial channel.

3. Bridge Approaches

On small streams, the route of the road should not be changed within the limits of bridge passages. On average and large rivers, this condition is difficult to meet; every effort should, however, be made to ensure that the bridge approaches deviate as little as possible from the general direction and form a continuous straight line with the axis of the bridge.

The route of the road within the limits of the stream channel should be as near as possible to a straight line and, in the case of a change in the road direction, the largest possible bend radii should be applied. Bends with small radii limit visibility to the detriment of traffic safety on the approaches.

Bridge approaches should not have a steep gradient, which decreases road capacity and is dangerous for vehicles. During periods of frost, the necessity may arise, on steep sectors of a road, completely to stop traffic.

On approaches with sharp gradients bends with small radii just before the bridge are particularly dangerous for traffic. Such sectors are often the scene of road accidents, particularly in winter, when descending the steep gradient at low speed is very difficult.

Considerable trouble is caused also by sloping approaches located within the limits of built-up areas. Narrow streets and steep slopes very often make it impossible for designers to plan approaches to conform with technical requirements without the removal of adjoining buildings.

If wide valley flats with old bayous or marshes are crossed by the road, large radii of bends turned upstream with their bows should be used to avoid such obstacles.

In particularly difficult cases, it is necessary to design a variant providing for a complete passing over of the obstacles and, in built-up areas, to plan a road outside the limits of the terrain covered with buildings.

4. Headroom Under a Bridge Above the Water Surface

The headroom under a bridge is counted from the water level for which the bridge opening has been computed, or from the highest level at which the river is still navigable and fit for rafting.

A vertical clearance, or the distance between the bridge structure and the water level, has been established by bridge building regulations. Some of the clauses of these rules are as follows:

(1) The lower structure of a bridge on rivers which are unnavigable and not fit for rafting must be raised 0.5 m. above the highest water elevation level for which the opening of such bridge was computed. On rivers where logs, bushes and other objects are expected to drift with highwater, this clearance should be increased and amount to 1.0 m.

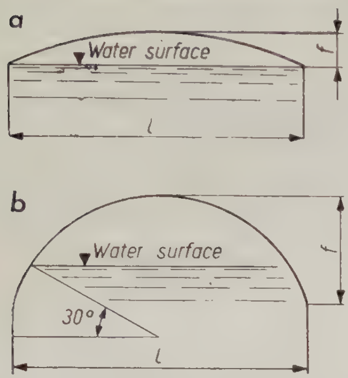

Fig. 83. Clearance of the bridge arch constructions above water level:

a for $\frac{f}{l} < \frac{1}{6}$, b for $\frac{f}{l} \geqslant \frac{1}{6}$

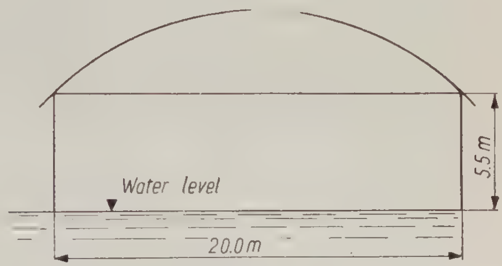

Fig. 84. Width of a navigable pass under an arch bridge

(2) The highest headwater level, for which were computed openings of bridges made of stone, concrete and reinforced concrete with roadway above should not reach above the arch base if the rise f of an arch does not exceed $\frac{1}{6}$ of its span (Fig. 83a):

$$\frac{f}{l} < \frac{1}{6}$$

If $\frac{f}{l} \geqslant \frac{1}{6}$, the level of a headwater may reach up to the height determined by a straight line inclined at an angle of 30° to the horizontal and traced across the center of the arch forming the intrados (Fig. 83b).

(3) The bearing on which the beams of the bridge are mounted and the coping of the piles of wooden bridges must be located at a height of 0.5 m above the headwater level adopted in the computation of the bridge opening.

(4) On rivers often used for rafting the bottom surface of the bridge construction must, when rafted timber is in the form of bound rafts, be elevated at least 2.5 m above the highest water stage at which river

traffic can operate and at least 1.0 m when logs are drifted as a loose mass. At the same time, it is necessary to ensure that all the above mentioned conditions are observed.

(5) The bottom surface of a navigable bridge span construction must be elevated at least 5.5 m above the highest water stage, at which vessel traffic is still possible. This elevation should be observed for each navigable pass over a length of at least 20 m. (Fig. 84). The observance of conditions in points 1, 2 and 3 is to be simultaneously checked.

(6) The highest water stage at which a river is still navigable is to be computed by the following method.

A table is prepared consisting of three columns. Numerals are put in the first column, the highest water stages observed according to their decreasing order in the second, and years in which they occurred in the third.

The effective water stage at which navigation may still take place is read on the line of the table, whose ordinal number N of the first column is determined by the formula:

$$N = \frac{n}{10} + 0.5$$

where n — number of years of observations taken into account by the table.

If the numeral determined by this formula consists of a full number and a fraction (e. g. 9.1), it should be rounded up to the full number (e. g. 10), which indicates the numeral of a line valid for the determination of the highest navigable water stage.

For canalized (stepped) rivers, where navigation is safeguarded by weirs over the whole navigable period, an artificially elevated stage, taking into account the curve of elevation, is recommended by the Soviet GOST 3035-45 regulations as the highest navigable water stage. If navigation on canalized rivers takes place during rises through open sluices, the highest navigable water stage to be adopted is that equal to analogous values accepted for uncanalized rivers.

The smallest outline of space, which should be left free for passing vessels and rafts, situated at right angles to the direction of the water current in every navigable pass, is called the under clearance of a bridge. No elements of the bridge passage or additional devices must be allowed to protrude inwards on this space.

In view of the development of navigation envisaged, and the planned increase in the transportation of freights, the necessity will probably arise in Poland to increase in some sectors for vessels passing bridges, the under clearance which has been in force thus far.

For this reason, the long-term planned development of water transport should be taken into account in building stable bridges.

Larger under clearances than those used up to now should be applied forthwith.

CHAPTER VII

RIVER REALIGNMENT IN THE VICINITY OF A BRIDGE

1. Deformation of the Shape of a Channel

All streams have a tendency to scour the surface across which they flow, to transport the scoured particles of ground and to deposit them at some distance from the place of scour.

Thus there occurs a deformation of the shape of the channel which should be taken into account in designing bridge passageways.

The process of deformation of the shape of the channel should not, however, be completely stopped, but efforts should be made to use the action of the river for improving flow conditions within the limits of a bridge passageway. This may be achieved by building such devices and auxiliary structures as will facilitate the river to form its channel within the bridge passageway according to the shape and dimensions designed.

The scour of ground can only take place when the velocity of the water current is higher than the velocity admissible for the ground forming its stream channel. The magnitude of the admissible velocity depends on the type of soil and on the water depth in a channel.

The bridge passageway can be designed without contracting the stream, for instance by building a single-span bridge across the whole width of the valley flat during the floods. If the bridge piers are located in the stream channel, local scours may occur near the piers, while the general character of the water movement remains unchanged.

For economical reasons, it is very seldom that a bridge is designed to cover the whole valley flat; as a rule its length is smaller than the width of such valley. In this case contraction of the stream takes place causing an increase in the velocity of the water under the bridge.

A considerable increase in velocity may prove fatal for the foundations of a structure and, therefore, the length of a bridge can be diminished only within certain reasonable limits.

An elevation and, consequently, an increase in the depth and a decrease in the gradient of the water level occurs as a result of the contraction of the stream. This contributes to a decrease in the velocity in an elevated area, while velocities also decrease along the river bed.

The regimen of the movement of sediment in a stream is disturbed by the

drop in velocity at the bottom. Some quantity of sediment brought by the river from its upper sector is deposited in the elevated area. For this reason, the quantity of sediment reaching the bridge opening is smaller than the amount which could be transported if there were no bridge.

The width of the water surface gradually diminishes in the vicinity of the inlet of the bridge opening, because the size of the opening is smaller than the width of the valley flat. For this reason a gradual increase takes place in the velocity of the water current in this sector (Fig. 85).

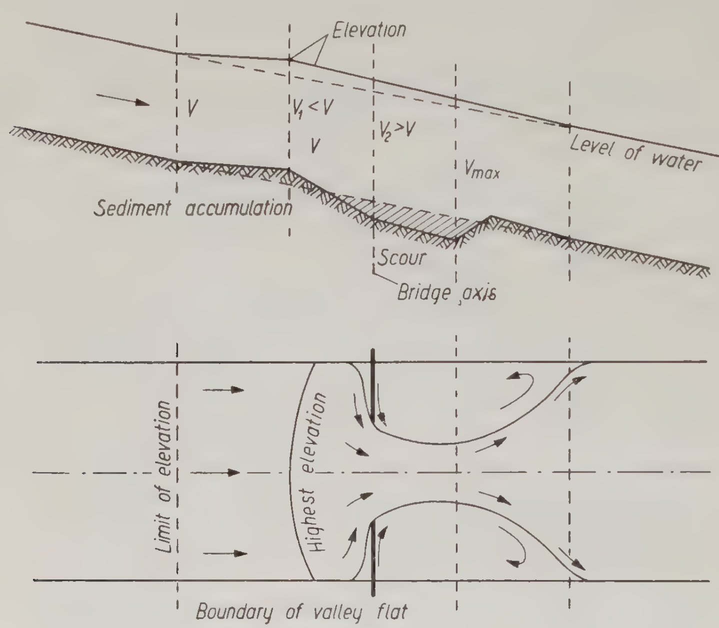

Fig. 85. Longitudinal section and top view of a stream within the limits of a bridge passageway;

The capacity of the stream for moving the suspended and bed material increases with a rise in the velocity of water. However, the quantity of sediment which reaches this sector is smaller than in a free channel and, therefore, only some part of the transport power of the stream is used for the carriage of such material.

An equalization in the quantity of material moved in this sector can take place only as a result of bed scour directly above the bridge. Scours of this type, increasing together with the decrease in the distance to the bridge, can be observed in practice.

The velocity of water movement increases under the bridge and directly below the bridge. This causes a bed scour and intensive movement of downstream sediment.

The streams of water, brought together after emerging from the opening, start to spread to the sides at some distance below the bridge and there appear simultaneously a decrease in the slope of the water level and in the velocity of the streams, particularly those flowing along the river bottom.

The transportation capacity of the stream decreases as a result of a decrease in velocity, and consequently the sediment is deposited in this sector.

The difference between water levels above and below the bridge is almost identical over the whole length of the bridge approaches. The water level along the bridge approaches is horizontal, and it assumes a considerable slope only at a very short distance from the bridge opening (Fig. 86).

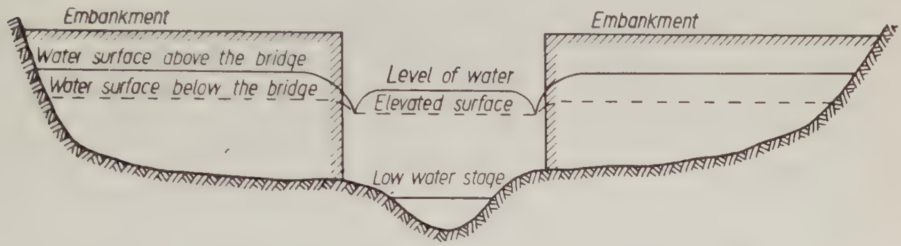

Fig. 86. River cross section within the limits of a bridge passageway

In a sector with a large slope of the water level, the velocity of flowing water currents increases markedly along the embankment on the side of the opening and, therefore, considerable scours usually appear in these places.

The velocity of these currents is insignificant a little further along the embankment, and sometimes even back currents arise there causing accumulation of sediment carried by the river to this area.

A break in the free surface of the stream appears in the immediate vicinity of the embankment cone (Fig. 86). Therefore, the slope of water level approaches zero value near the declivity on the side of the tailwater and sometimes, near the bridge, it even assumes a negative value, the currents flowing in such a place in the direction of a bridge opening. The slope also causes currents from the valley flats to start to flow towards the bridge, joining the stream flowing out of the opening.

The contraction of the stream, whose cross section assumes the lowest value along the whole sector of a bridge passageway, takes place at some distance below the bridge. The distance from the axis of a bridge to the cross section fluctuates within limits of $\frac{1}{2}$ and $\frac{1}{5}$ of the size of the bridge opening. The stream gradually widens below the cross section.

The contracted cross section of the stream described above is a place where the mean water velocity reaches its peak and, there the largest channel deformations are observed within the limits of a bridge passageway.

The changes extend to some distance down- and up-stream from the cross section, sometimes reaching as far as under the bridge — a factor to be taken into account when designing the foundations of bridge piers. The extent of the channel deformation within the bridge passageway depends on the duration and magnitude of a rise. The largest changes in the shape of the channel occur during the water rise; they decrease slightly during the highest water stage, do not appear at all during the fall of water, and sometimes there is an accumulation of sediment instead of erosion. It is possible for the main channel to remain silted for quite a long time after the passage of a high water before it regains its former state, while this cannot fully take place in valley flats because water flows off these areas directly after the rise.

The scour and sedimentation of the main channel will take place during each subsequent rise, while the terrain changes of valley flats will increase after each rise.

An uneven distribution of velocities, which assists the formation of local scours in the bridge cross section of water is caused by a marked distortion of streams under the bridge. These scours are particularly dangerous in the vicinity of bridge piers.

The deformations of the channel within the bridge passageway depend, therefore, on the mean velocity of water movement, and the longitudinal slope of the water level of the stream and distribution of velocities on the width of the opening and the quantity of sediment brought or taken away by the river.

2. General Methods of Protecting a Bridge Passageway Against the Dynamic Action of Water and Ice

A change in the stream regimen takes place within the bridge passageway, causing in this area channel deformation, which may contribute to a diminution in the stability of bridge piers. Furthermore the streams flowing with a high velocity in the narrowed sector of a river can directly damage the structures situated within the limits of the bridge passageway.

The methods of combatting the damaging action of a stream may be divided into two groups. Group I — passive methods and Group II — active methods. A combination of both these methods can also be applied.

The passive method of combatting the destructive action of the stream consists in changing the shape and reinforcing the construction of a structure so that a stream cannot damage it. Active methods include building additional devices to cause a change in the regimen of the stream. Efforts should be made to ensure that these devices contribute both to an increase in the structural stability, and to facilitate the utilization of water power for the useful work of shaping the channel.

At first, only passive methods were used for protecting structures against

the destructive action of water streams. They consisted in the reinforcement of bridge constructions — primarily of all those parts exposed to the direct action of flowing currents. In addition, the depths of founding bridge piers were increased and the river sectors in which considerable deformations might take place were reinforced.

In 1872, an active method of combatting the destructive action of water was applied in Russia for the first time. Special structures were then built, directing the streams under the bridge on the Kvirila River, to remove dangerous scours which arose in the valley of that river after a bridge and its approaches had been built.

A few years later, similar structures were introduced on the Western Bug River, the Dnieper River in Kiev, etc.

River Training Works within the Limits of the Bridge Passageway

In building a bridge passageway, river training structures should be adapted for periods in which the water stages in a stream are low and for periods characterized by high stages.

Some of such structures are located in the direct vicinity of a bridge, to create the most favorable conditions possible for water flow passing through the bridge opening. Another part of the river training structures is located in the river at some distance above and below the bridge.

The river training structures intended to improve flow conditions during low water stages are: transversal and longitudinal dikes, or groins, rapids, cuttings, reinforcements of banks against damage and bridge piers against scours, floating devices serving to change the intensity and direction of streams, etc.

Such devices serve for deepening or silting some parts of a river, removing or fixing channel deformations, removing side branches, decreasing the oxbows in a main channel, straightening the river within the limits of a bridge passageway, etc.

Smaller or greater rises of water stages may appear during certain periods. They are extremely dangerous for the stability of structures, although their duration is usually short. These rises may cause considerable changes in the shape of the channel, because high current velocities then arise, the motive force of the stream is increased, and great transverse water circulation appears in a river.

River training structures whose shape may vary with local conditions, are built within the limits of a bridge passageway to improve the conditions of water flow during rises.

Directional dikes, unflooded during rises, are the principal type of such structures. They assist the streams to increase the velocity gradually without any turbulences and jumps, and decrease the intensity of the transverse water circulation in the vicinity of the bridge opening.

Groins and all types of bank reinforcements are applied to protect bridge approaches located in valley flats.

Special care should be given to designing river training structures within the limits of a bridge passageway, because the cost of building such is high, even in some cases higher than the cost of building the bridge itself. Accurate methods of computing particular types of river training structures have not so far been elaborated, in view of the complex character of phenomena occurring during water discharge in a scoured channel.

Therefore, only very approximate results can be arrived at by using all the simplified methods of computation applied to the designing of river training structures.

To investigate a bridge passageway model in the laboratory is, therefore, the most rational method.

Bottom Scour Near Bridge Piers

The cross sections of pier foundations are usually rectangular in shape — therefore easier to make — while above the offset of the foundation piers are semistreamlined.

A local ground scour often appears in the vicinity of piers, which is dangerous for the stability of the structure, whereas the scour caused by rectangular piers is 10-20 percent greater than that caused by semistreamlined piers.

To diminish the depth of the ground scour near the bridge piers, the semistreamlined shape of the foundation footing should also be applied at a depth of 1 m below the level of possible scour. The front and back parts of the watertight walls surrounding the pier foundations should taper at an acute angle.

The depth of the stream after the scour near the pier with a semistreamlined foundation can be determined by the following formula presented by Boldakov:

$$h_r = hP\left(\frac{v_1}{v_o}\right)^{1/4}$$

where:

h_r — depth of stream near the pier after the scour, in m,
h — depth of stream near the pier before the scour, in m,
P — coefficient of scour adopted within limits of 1.0 and 1.4,
v_1 — water velocity in a main channel, in m/sec,
v_o — water velocity at which ground particles cease to move, in m/sec.

The depth of the scour near the piers with a rectangular cross section can also be computed by this formula. The exponent of power $\frac{1}{3}$, instead of $\frac{1}{4}$ as adopted for the semistreamlined piers, should be applied in this case.

If a slanting pier is located at an angle larger than 10°, the scour near it may be computed by the same formula in which the exponent $\frac{1}{2}$ is introduced.

It should be remembered that water velocity v_o, at which ground particles cease to move is 1.3 times lower than the velocity v_n, which does not cause scour. Velocities causing no scour at a depth of 1 m are presented in Table 73. If water depth H is less or greater than 1 m, the velocity v_n' causing no scour can be computed from the formula:

$$v_n' = v_n \sqrt[5]{H}$$

The local scour near pile strutting takes place in a way different from the scour near massive piers.

A funnel-like pit is formed near each pile, and its depth t in m can be computed by the Latishenkov formula:

$$t = av \sqrt{\frac{d \, tg \, \varphi}{v_n H^{0.2}}}$$

where symbols denote the numerical values of individual magnitudes:

d — diameter of pile, in m,
v — mean velocity in vertical near the pile, in m/sec,
v_n — mean velocity causing no scour, in m/sec (Table 73),
H — stream depth after the scour, in m ($H = h + t$),
φ — angle of inclination of the pit slope around the pole, equaling the angle of natural wet ground (for sand $\varphi = 30°$),
a — coefficient of proportionality $= 0.65$.

The magnitude of the local bottom scour computed by the formulas given above should be taken into account in designing the depth of foundation piles. If the depth is too large, an effort should be made to diminish the change in the shape of the channel by means of various additional methods.

Table 73

Velocities v_n which do not Cause Scour in Loose Grounds at Water Depth $H = 1$ m

Diameter of particles in mm	v_n in m/sec
0.05	0.35
0.25	0.50
1.00	0.60
2.50	0.70
5.00	0.85
10.00	1.00

Scour may be diminished, for instance, by extending the span of the bridge, bracing the stream bottom near the piers, by river training structures, etc.

If the foundation of a pier is shallow, a sheet piling single or double, is driven into the ground around the foundation as a protection against scour. Such a reinforcement is sometimes insufficient because the ground around the sheet piling may also be scoured, and then the sheet piling loses its stability.

A rock filling consisting of stones 25-50 cm in diameter is also applied to protect the ground around the piers against scour. As the ground is scoured

and the filling settles, a certain quantity of stones must be added. The costs of a rock filling are very high. Moreover, a reinforcement of this type causes contraction of the cross section of the stream under the bridge, which may result in an additional scour.

Brush fascine mattresses loaded with stones might be used instead of rock filling. The thickness of a mattress may reach 50-70 cm, and of a loading layer — 35 cm. The width of a mattress should not be less than 10 m on each side of a pier. Directly near the pier, the mattress is loaded with a stone layer 1 m wide (Fig. 87a).

The piers located in the valley flat cannot be reinforced with fascine mattresses because the water may dry up and destroy the material. Concrete slabs and single or double pavements are, therefore, recommended to brace the piers in valley flats. The width of a paved sector on each side of a pier should be larger than 5 m (Fig. 87b).

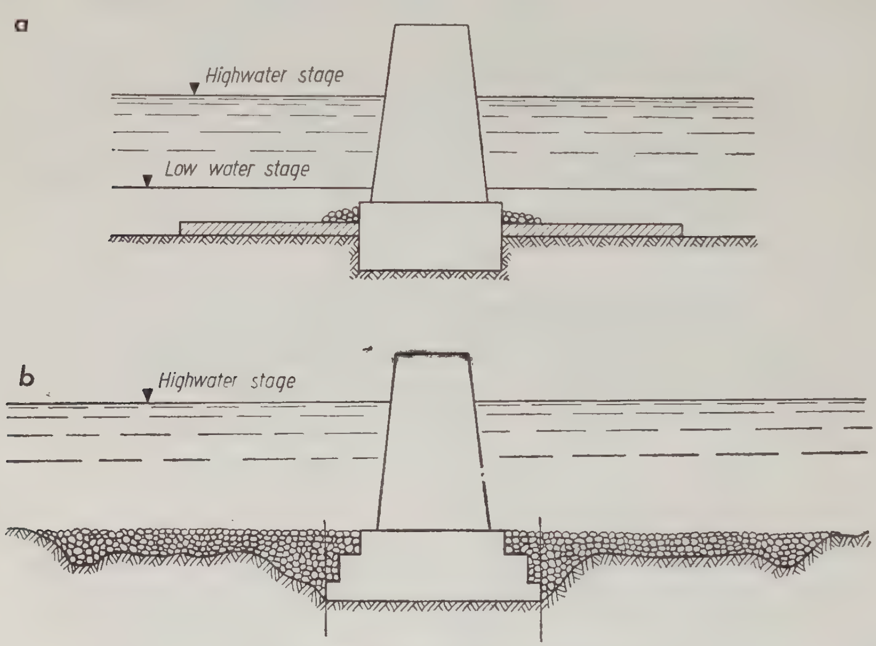

Fig. 87. Protection of bridge pier against scour: *a* — brushwood mattresses, *b* — paving

Protection of Bridge Piers Against the Action of Ice

In Poland, all rivers are usually covered with ice in winter, and the small rivers, of course, freeze earlier than the large. The freezing of rivers mostly occurs at the beginning of December.

The thickness of ice cover depends on winter temperature, thickness of snow cover, etc. Rivers with higher water velocities have a thinner ice cover.

In Poland, the thickness of ice cover fluctuates on an average within limits of 0.3 and 0.7 m.

Ice cover usually freezes to the banks. During the changes of water stage, the ice cover bends or breaks away from the banks. When water rises, the timber piles of the bridge piers frozen to the ice cover may be torn from the ground. That is why the ice should be hewn around the pile strutting.

In spring, in this country generally at the end of March, the ice cover breaks and the ice drifts. If the snow thaws slowly and the temperature gradually rises, an ice drift appears at comparatively low water stages. If there is a sudden rise in temperature with simultaneous rains and warm winds, the snow melts very quickly and water running off the still frozen ground may cause a great rise. Ice drifting on the crest wave of the flood may damage bridge constructions.

Upon encountering any obstacle, ice floes stop and accumulate, piling up and forming an ice gorge rapidly spreading upstream. An ice gorge thus formed, sometimes several meters high, bars the river and causes its valley to be flooded.

If the river banks are high and the barrier is formed over the whole width of the river, the water level rises. When, consequent on the action of water pressure and great velocities of water flowing among the ice floes, a breakage of the ice gorge occurs, vast quantities of water with ice floes flow downstream, destroying all structures and other objects encountered on their way.

If the banks of a main channel are low and of soils with low resistance, water may find its way along the bottom under the ice gorge, or across the valley flats, and destroy the bridge approaches.

The formation of an ice gorge may be caused by a bridge opening being too small, inappropriate location of a bridge, incorrect distribution of river training structures and icebreakers, etc. Ice gorges are also formed in sharp river bends, on rapids, contracted channels, etc.

To prevent the formation of ice gorges, the river channel should be straightened, its bottom cleaned, and the structures should be appropriately distributed along and across the river. Then the ice floes will flow freely, causing no damage.

Icebreakers are often located in front of bridges to facilitate ice drift under them and to protect pile strutting against damage.

The placing of icebreakers in front of wooden bridges on rivers with intensive ice drifts is necessary because of the low resisting strength of pile struts and, therefore, wooden icebreakers, flat or wide, are designed to suit the construction of the pile strutting.

Boldakov[5] suggests that an icebreaker should be connected with a pile

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[5]) Boldakov — "Perekhody cherez bolshye vodotoki" (Bridge Passageways Across Large Streams). Moscow 1949, p. 188.

strutting thus increasing its stability. According to his numerous observations of ice drifts through the openings of wooden bridges, ice floes striking the icebreakers connected with pile struts cause insignificant shocks to the bridge structure, and there is no reason to fear the negative influence on the stability of a wooden bridge said to be exerted by joining the icebreaker with a pile strutting.

A more streamlined shape to a pier is obtained by building icebreakers connected with pile struts. On the other hand, eddies are formed by separately built icebreakers, which cause the stream to become narrow around the piers, thus diminishing the capacity of a bridge opening by 5-10 percent.

In view of the above remarks, Boldakov recommends the application of the following guiding rules in designing icebreakers and bridge pile struts:

(a) wooden struts should be joined with icebreakers by means of a common cover for the two elements; large icebreakers should be filled with stones and the back part of the pile strutting should be semistreamlined;

(b) the cutting, front edge of the wooden icebreakers should have a 1 : 1 to 1 : 2.5 inclination depending on the intensity of ice drifts;

(c) if cribwork is used piers should be designed with sharp endings at an angle of at least 90°.

When designing icebreakers, ice drifting through the opening of neighboring bridges should be observed and the dimensions and shapes of those

Table 74

Numerical Data for Designing Icebreakers

Intensity of ice drift	Ice thickness in m	Inclination of the front edges of icebreakers		Elevation of upper parts of icebreakers above water level in m		Lowering bottom parts of icebreakers below the lowest level of the ice drift in m	
		massive	wooden	massive piers	wooden piers	massive	wooden
Very high	1.0	1 : 0.5 to 1 : 0.1	1 : 3 to 1 : 2	1.0	1.5	1.0	1.2
High	0.8	1 : 0.1	1 : 2.5 to 1 : 1.5	1.0	1.2	—	1.0
Average	0.6	1 : 0.1	1 : 2 to 1 : 1	0.75	1.0	—	0.7

icebreakers which best serve their purpose should be adopted in the future structure. If it is not possible to make such observations, the data presented in Table 74 may be used for the purpose.

The dynamic pressure P in tons of ice on a pier may be determined by the following formula:

$$P = ahB$$

where:

a — ice pressure in tons per 1 m of the width of pier with ice thickness = 1 m; ice pressure a is usually assumed as 75 tons/sq m and, for high ice drift, as 50 tons/sq m;

h — thickness of ice, in m;

B — width of pier, in m; for piers wider than 2 m, half the width of the remaining part should be added to this width.

The point of application of force P is assumed as being at the height of an ice drift.

3. Development of an Artificial Transverse Circulation of Warer

In a stream, there arise various velocities of the surface and bottom streams of water, consequent on the action of centrifugal and friction forces. The transverse circulation of water is caused by these velocities.

The transverse circulation of water occurs, for instance, in the curved sector of a river, where centrifugal forces seeking to drive the water particles from the convex to the concave bank act on such particles.

The longitudinal velocities of water current on the surface are greater than near the bottom and, therefore, a greater centrifugal force acts on the upper layer of water than on the bottom layer. The upper layer is driven by a greater force to the concave bank, near which it falls to the bottom, pressing through to the lower layer on which a smaller centrifugal force acts, driving it to the convex bank. This layer reaching the concave bank is raised to the surface.

In touching the concave bank, the surface currents scour it. Dropping to the bottom, these currents take with them the ground particles from the scour which, subsequently, are drifted in suspension to the convex bank following the action of the transverse circulation of water (Fig. 88).

The longitudinal velocities are greater at the concave and smaller at the convex bank. Therefore, the channel at the concave bank is scoured and ground particles are carried along the bottom to the convex bank where they are deposited.

Thus, the deformation of a stream channel is caused by the transverse circulation of water. Attempts were made, by means of appropriate devices, to form such a direction of transverse circulation in a river as to stop the harmful deformation of the channel and force the streams of water to perform the work necessary to improve conditions of discharge.

This was achieved by means of various types of shields, suitably located in the streams.

By introducing artificial transverse circulation, banks can be protected against scour, meanders straightened, the bottom deepened, intensity of silting diminished, and the bottom secured against scour near the bridge piers.

A bridge pier divides the river into two parts, which unite again behind it (Fig. 89). The bottom streams approaching the piers are directed downwards and outwards from the pier, thus causing the scour.

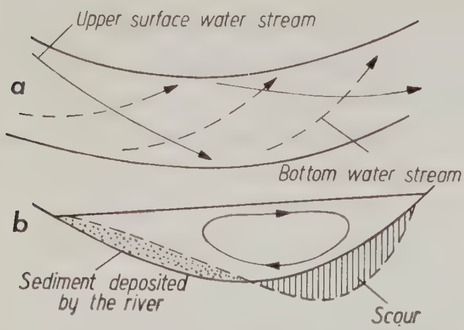


Fig. 88. The change in the shape of a channel along a river bend: *a* — top view, *b* — cross section

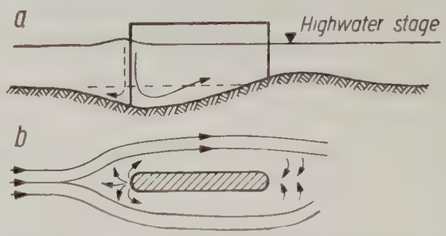


Fig. 89. Water discharge near the bridge pier: *a* — cross section, *b* — top view,

Sediment accumulates behind the pier, because streams encircling the pier on either side come together.

To avoid the possibility of such deformation of the channel, the direction of currents should be changed by special directive dikes causing artificial transverse circulation of water.

Potapov recommends protection of the bridge piers by means of two bottom directive systems made of fascine (Fig. 90). The height of such a directive dike is assumed at $h = 0.3—0.5\ H$ and length $l = 2.5—3\ H$, where H — depth of water near the pier during the low summer water stages. The angle α contained between the axis of a directive system and the longitudinal axis of a stream is adopted within limits of 12-18°.

To increase the quantity of the material deposited, nets, shrubs, etc. are placed between the pier and the directive system.

Surface or internal directive systems in the form of shields attached to piles are applied on weak, loose grounds.

4. Water Directive Dikes

A bridge constitutes an obstacle to the free flow of water in a stream and can fundamentally change the regimen of such stream, particularly at high water stages.

Various river training structures are often built to improve discharge conditions within the limits of a bridge passageway. Among the most important of these are directive dikes.

Directive dikes are divided into upper and lower, depending on which side of a bridge (along the river) they are located.

The upper directive dikes (leading in), which are located above the bridge

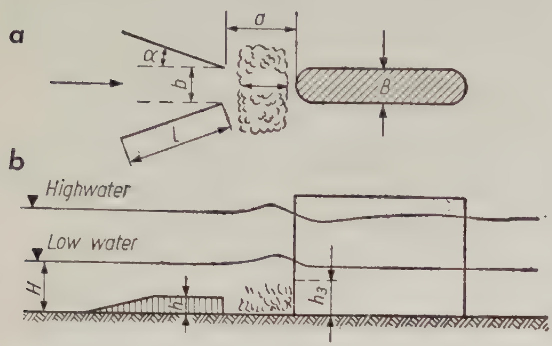


Fig. 90. Protection of bridge pier against scour
a — top view, *b* — cross section

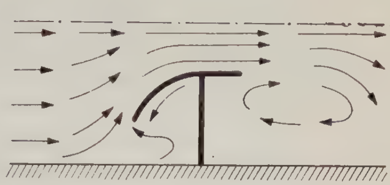


Fig. 91. Water discharge in a channel near a directive dike

in prolongation of the abutments, introduce the streams of water from valley flats into the openings of a bridge during rises (Fig. 91).

The lower directive dikes (leading out), located similarly but below the bridge, protect the currents outgoing from the opening against the influence of water on both sides of the valley flat. For this reason, streams flowing out of the opening in a valley flat gradually unite at a considerable distance from the bridge.

The most protruding part of a directive dike above the bridge is called the head. The slope of the wall on the side of the main channel is called the inner, wet or river slope, and on the side of the valley flat — the outer, dry slope of the valley flat.

The directive dikes built in the crossings of lowland rivers are usually of considerable height and therefore broad of base. They are not, however, exposed to the action of headwater, since the difference in levels on the two sides of the wall is insignificant. Directive dikes on lowland rivers are therefore built in the form of an embankment with trapezoidal cross section. The inclination of the slopes as adopted amounts to 1 : 2.

The width of the crown of a directive dike is not usually smaller than 2 m so as to facilitate a one-directional transport of the building materials necessary to reinforce or repair sectors which may be damaged by rises.

The crown of the dike is raised 0.5 m above the water level adopted for computing the bridge, taking into account the elevation above the bridge and the height of wave reaching the slope of the embankment.

With directive dikes, the water surface near the upper slopes of bridge approaches in valley flats does not have sectors as described above with sudden slopes, since the dike prevents a direct spilling of water into the bridge opening.

A change in the conditions of the movement of the stream consequent upon construction of directive dikes causes a different distribution of scouring and silting of the bottom within the limits of a bridge passageway (Fig. 92).

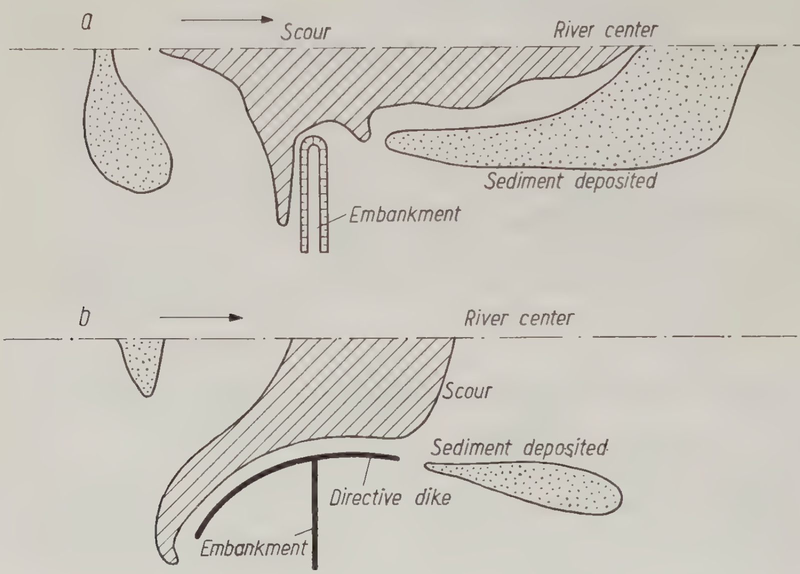


Fig. 92. The change of the shape of the bottom under a bridge:
a — channel without realignment structures *b* — channel with a directive dike

Properly designed directive dikes change the movement regimen of the bottom material, diminishing bottom scour along the axis of the stream and in the vicinity of the approaches. Directive dikes which enable the size of the opening to be diminished should always be built within the limits of a bridge passageway. The contraction of a stream below the bridge can be entirely avoided by means of directive dikes, which may also help to diminish the deformation of the channel.

If the amount of water afflux from the valley flats does not exceed 20-30 percent of the entire volume of disharge, the construction of directive dikes is not necessary, because the contraction of the stream is small and no major changes in the shape of the bottom can be expected.

The directive dikes may have a recti- or curvilinear shape. Diagrams of directive dikes of rectilinear, curvilinear with a straight sector, and fully curvilinear shape are shown in Fig. 93.

At first, rectilinear directive dikes were recommended, so that the conditions of discharge under the bridge were similar to the conditions of a uniform movement in a straight channel.

However, in the event of rectilinear directive dikes not being long enough, the central part of the bridge opening is overloaded, which causes a considerable scour of the bottom and the deposit of ground particles in the extreme parts of the opening.

Curvilinear directive dikes have proved to be the best. At first, they were applied in the form of two circular arcs connected by a straight line. Later on, circular directive dikes were built without the straight sector.

It has now been proved that the best results are obtained with walls having the shape of a false ellipse consisting of several arcs whose radii increase towards the bridge.

It has also been recommended, on the basis of laboratory tests, to use upper directive dikes elliptical in shape and the long semiaxis directed perpendicularly to the axis of the bridge passageway.

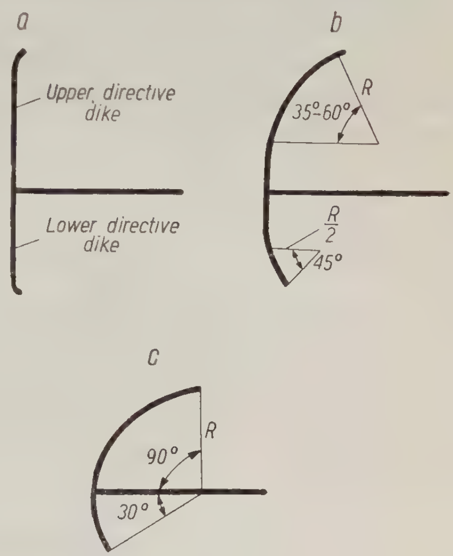


Fig. 93. Scheme for Directive Dikes
a — rectilinear, *b* — curvilinear with straight sector, *c* — fully curvilinear

Curvilinear directive dikes should meet the following conditions:

(1) The curvature of the dike directly at the bridge should be very small or even equal to zero, because it is necessary that all the currents of a river be led to the straightened opening.

If the currents passing through the bridge opening are straight and parallel, the stream is not exposed to contraction below the bridge. Currents of such a river emerging from the opening spread gradually and uniformly.

(2) The outline of a dike should be in the shape of a curve of variable curvature because:

(a) streams in valley flats, flowing towards the bridge parallel to the approaches, must by means of the directive dikes change direction at right angles;

at first, therefore, the current should be curved by the directive dikes and subsequently straightened again. Thus the curvature of the dike must be considerable at some distance above the bridge, because the curving and turning of slowly flowing currents takes place in this sector; on the other hand, the curvature of the dike immediately in front of the bridge opening should be small, because it is there that currents whose velocity was increased during the general contraction of the river are straightened;

(b) directive dikes of variable curvature, steadily changing with the approach to the bridge opening, cause transverse circulation of low intensity in a river which transports eroded material to considerable distances from the bridge; passing from the narrowed to the normal cross section takes place gradually, and therefore no channel deformation occurs in the form of concentrated scours.

(3) Directive dikes with large curvature in the vicinity of the bridge opening are not satisfactory, because they do not counteract the contraction of the river below the bridge.

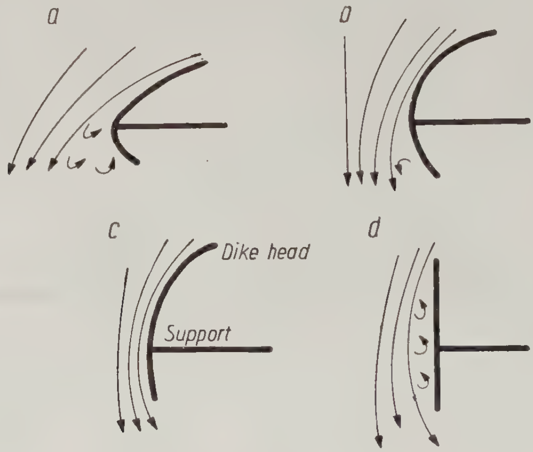


Fig. 94. Stream directions near directive dikes of various shapes

Immediately at the bridge, the streams deviate from such types of directive dikes; the extreme parts of the bridge opening barely participate in passing the water, and the shape of the channel changes.

Dikes with a large curvature in the vicinity of a bridge cause a very intensive transverse circulation in the river. Particles of scoured ground then accumulate directly below the bridge opening and are deposited in the side parts which do not participate in passing the discharge.

Directions of the currents near directive dikes of various shapes are shown in Figure 94.

In addition to curvilinear directive dikes, rectilinear dikes are also used in practice, although the most suitable shape for such is a line of variable curvature.

Selection of the outline of a directive dike may be facilitated in particular cases by using Table 75, elaborated on the basis of theoretical considerations, field studies of directive dikes, and laboratory tests.

Table 75

Selection of Outlines of Directive Dikes

Characteristics of a stream and valley flats	Shape of directive dikes in a main channel	
	right bank	left bank
Two-sided valley flats		
The right hand side of valley flat works more intensively than the left:		
(a) rectilinear stream	curvilinear	rectilinear
(b) the right bank is convex, the left bank — concave	curvilinear	rectilinear
(c) the left bank is convex, the right bank — concave	rectilinear	curvilinear
The left hand side of valley flat works more intensively than the right:		
(a) rectilinear stream	rectilinear	curvilinear
(b) the right bank is convex, the left bank — concave	rectilinear	rectilinear
(c) the left bank is convex, the right bank — concave	rectilinear	curvilinear
Two-sided even valley flats		
(a) rectilinear stream	curvilinear	curvilinear
(b) the right bank is convex, the left bank — concave	curvilinear	rectilinear
(c) the left bank is convex, the right bank — concave	rectilinear	curvilinear
One-sided valley flat		
(a) rectilinear stream	curvilinear	reinforcement of the cone of a high bank
(b) stream curved towards the high bank	curvilinear	reinforcement of the cone of a high bank
(c) stream curved towards valley flat	rectilinear	reinforcement of the cone of a high bank

Tracing Water Directive Dikes and Determining Their Shape

Table 76, where values of the coordinates x and y, radii of curvature ρ, and lengths of dike sectors ΔS are given for various values of the angle α, was

elaborated for tracing the outline of the upper and lower directive dikes in a terrain. The starting point of the system of coordinates was located at the point where the upper and lower directive dikes contact the bridge abutment (Fig. 95).

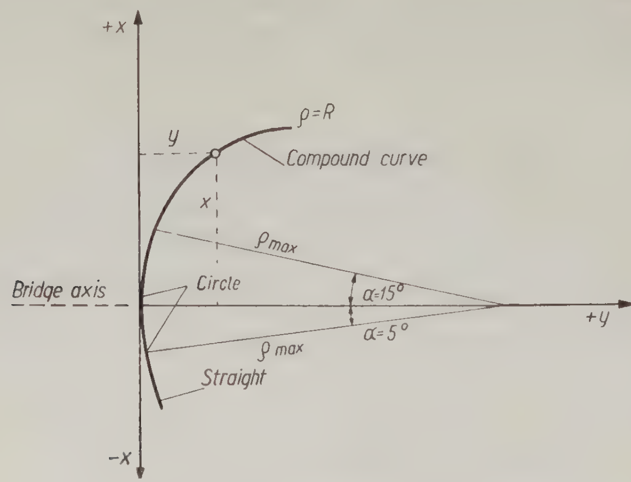


Fig. 95. Tracing the directive dikes

Table 76 presents values for two cases:

$$v_d = v_m = \text{const}$$

$$v_d = v_m \sqrt{\cos \alpha}$$

where:

v_d — velocity of current flowing along the directive dike,

v_m — velocity of an extreme current in a bridge opening.

The velocity of a current flowing along the directive dike may be constant (v_d = const.), equaling the velocity of an extreme current in a bridge opening, or it may increase along the wall. The initial velocity of a current near the head of an upper dike may even be equal to zero. At the end part of a dike near the bridge abutment, the velocity v_d is always equal to the velocity v_m.

The velocity may change in a different way along the directive dike. It is generally assumed that velocity increases according to the following correlation:

$$v_d = v_m \sqrt{\cos \alpha}$$

Then, at $\alpha = 90°$ near the dike head, the velocity $v_d = 0$, while at $\alpha = 0$ (near the bridge abutment) $v_d = v_m$.

The values in Table 76 for two cases of increase in velocity can be considered as extreme, and between them are included all other values encountered in practice. Since the lengths of directive dikes for these two extreme cases differ

Tracing Curvilinear Directive Dikes

Velocity			$V_d = V_m = \text{const}$				$V_d = V_m \sqrt{\cos\alpha}$			
Radius of curvature			$\rho = \frac{R}{\sin}\alpha$				$\rho = R \operatorname{ctg}\alpha$			
Directive dike	shape of dike	$\alpha°$	$\frac{X}{R}$	$\frac{Y}{R}$	$\frac{\rho}{R}$	$\frac{\Delta S}{R}$	$\frac{X}{R}$	$\frac{Y}{R}$	$\frac{\rho}{R}$	$\frac{\Delta S}{R}$
Upper	Curve with variable radius	90	2.350	1.438	1.000		2.021	0.868	0.000	
						0.75				0.014
		80	2.336	1.262	1.015		2.020	0.853	0.176	
						0.182				0.048
		70	2.288	1.087	1.072		2.006	0.808	0.364	
						0.191				0.081
		60	2.207	0.914	1.155		1.973	0.734	0.577	
						0.216				0.123
		50	2.804	0.740	1.305		1.927	0.634	0.839	
						0.246				0.175
		40	1.909	0.566	1.555		1.777	0.511	1.192	
						0.307				0.252
		30	1.657	0.391	2.000		1.570	0.368	1.732	
						0.425				0.379
		20	1.278	0.216	2.921		1.219	0.210	2.747	
						0.279				0.278
		15	1.000	0.132	3.861		0.966	0.127	3.732	
	Sector of a circle					0.337				0.325
		10	0.672	0.058	3.861		0.651	0.056	3.732	
						0.337				0.325
		5	0.336	0.015	3.861		0.325	0.015	3.732	
						0.337				0.325
		0	0.000	0.000	3.861		0.000	0.000	3.732	
Lower						0.337				0.325
		−5	−0.336	0.015	3.861		−0.325	0.015	3.732	
	Straight					0.421				0.345
		—	−0.775	0.051	∞		−0.667	0.045	∞	
						0.421				0.345
		—	−1.175	0.088	∞		−1.010	0.075	∞	

Length of directive dikes	$V_d = V_m = \text{const}$	$V_d = V_m \sqrt{\cos\alpha}$
Upper	$S_g = 3.032$	$S_g = 2.325$
Lower	$S_d = 1.180$	$S_d = 1.015$
Total	$S = 4.212$	$S = 3.340$

only slightly, there is no necessity to determine other possible correlations relevant to the change of velocities of streams along the dike.

Figure 96 shows two extreme outlines of curvilinear upper directive dikes.

After substituting and supplementing from the equations $v_d = v_m$ and $v_d = v_m \sqrt{\cos \alpha}$, the magnitude of the radius of curvature can be obtained:

$$\rho = \frac{R}{\sin \alpha}$$

$$\text{and } \rho = R\text{ctg } \alpha$$

The parameter R determined by means of the hydraulic characteristics of the bridge passageway appears in these equations — namely:

$$R = \frac{Q}{Q - Q_z} \cdot \frac{W_z^2}{g}$$

where:

Q — full discharge in cu m/sec,

Q_z — discharge in cu m/sec existing formerly over the width of valley flats and covered subsequently by the bridge approaches,

W_z — modulus of the stream velocity in valley flats in m/sec equaling $C\sqrt{H_z}$,

g — acceleration due to gravity in m/sq sec,

H_z — depth in valley flats, in m adopted in computing the bridge opening.

The geometrical meaning of the parameter R can be explained to some extent by introducing the angle $\alpha = 90°$ to the equation $\rho = \frac{R}{\sin \alpha}$ and then, $\rho = R$. When $\rho = R\text{ctg } \alpha$, then $\rho = R$, with the value of the angle $\alpha = 45°$.

Using Table 76, the values obtained should be multiplied by the parameter R computed by the formula given above, because the values are presented in the Table for $R = 1$.

It should be borne in mind in tracing directive dikes, that the upper wall does not always need to be of such a length that the angle contained between extreme radii amounts to $\alpha = 90°$. If there are no streams flowing towards the opening parallel to the axis of the bridge passageway, we should confine ourselves to that angle α_{max} at which the extreme currents flowing towards the opening are inclined to the axis of the river.

In some cases, computations give very low dimensions of the directive dikes. Then, cones of embankments can be used to replace them. If the cones do not entirely cover the dikes, the height of the cones should be raised above the water level by adding material or by diminishing their inclination.

Sometimes, too great a length of directive dikes may be arrived at by computation. In that case, the most appropriate length of dike should be found by model tests, because the computation of dikes by theoretical formulas is not

always accurate. The shortest dike is selected by way of laboratory tests and it produces satisfactory conditions of discharge.

Application of Water Directive Dikes Under Special Conditions

The curvature of directive dikes near the abutment is small when the axis of a bridge is situated in perpendicular to the main channel and to the direction of the highwater flow.

In some cases, the bridge passageway is traced obliquely to the river channel. Then, another shape of directive dikes should be applied, best determined, because it is difficult theoretically to envisage how river streams would be distributed under the bridge, by laboratory investigations of a model of a bridge passageway.

When a river is crossed obliquely a long directive dike is usually designed on that side of the opening from which flows the principal mass of water (Fig. 97). The curvature of this dike at the bridge should be small or even equal to zero, because in the event of an oblique crossing the streams have a tendency to deviate towards one of the banks, and even with a rectilinear directive dike, the streams do not touch the dike over its entire length.

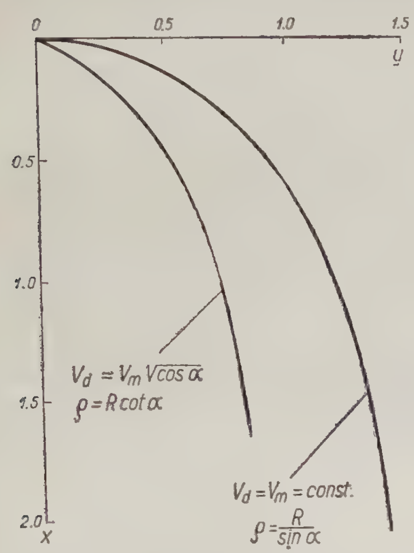


Fig. 96. Extreme position of directive dikes

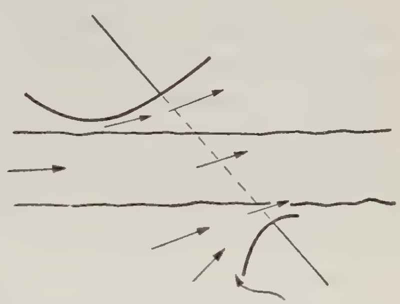


Fig. 97. Directive dikes at a stream crossing in a slanting direction

Short dikes with considerable curvature near the bridge are applied on the other side of an opening.

Directive dikes on rivers having side branches must be designed with particular care. On mountain rivers where river training is not recommended, large openings flanked by strong directive dikes, between which the river can change the situation of its channel, should be applied (Fig. 98).

If there is a necessity to divert the current from the bridge abutment, rectilinear directive dikes should be used. Directive dikes of this type are designed on rivers with variable beds or when one of the abutments is exposed to scour resulting from a change in the main channel.

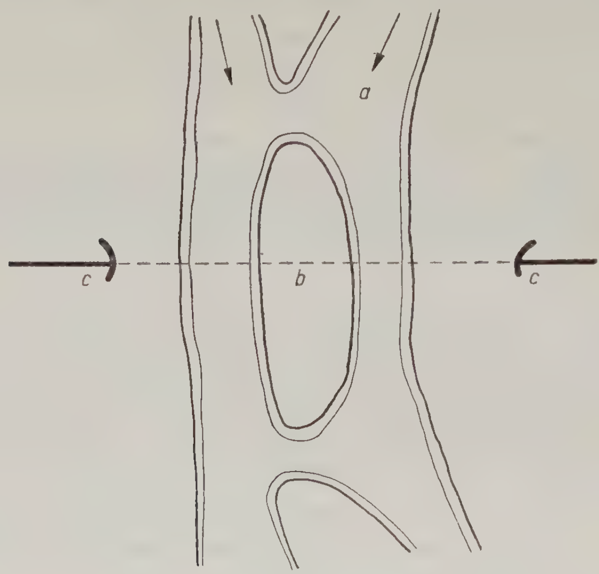


Fig. 98. Diagram of crossing of a two-branch river

Rectilinear directive dikes may be used on rivers with two valley flats if the deviation of the channel does not exert a negative influence on the elements of a bridge passageway.

The length of the rectilinear directive dikes — in the event of there being two valley flats — are usually above the bridge, adopted as a magnitude not lower than half the length of the bridge opening and, below the bridge — not lower than $\frac{1}{4}$ of the length of the opening.

Joining Water Directive Dikes with an Abutment

Bridge constructions are usually finished off with abutments and earth cones which end the slopes of the embankment forming the bridge approaches are built on both sides of the abutment. These cones may cover the abutment partially or fully. The water directive dike contacts the cone of the embankment.

Appropriate connection of the directive dike with the abutment is extremely important on account of the danger of infiltration which may occur along the line where the directive dike contacts the wall of an abutment and the earth cone.

Infiltration of water here is very dangerous, causing scour of ground particles, which may consequently result in the occcurence of a fissure through which water will be spilled directly from the valley flat into the bridge opening.

The directive dike should be smooth over its whole length on the side of a river, and should not have any protuberances hindering the movement of water. For this reason, the slopes of the directive dike on both sides of an abutment should be placed forward, towards the main channel, over such a width that the inner (front) edge of the wall crown can be located in the continuation of the wall of the abutment. The slopes of the directive dike on both sides of the abutment are connected under the bridge to facilitate the free flow of currents through the bridge opening in the vicinity of the abutment.

If vehicular traffic is envisaged on the crown of the directive dike, its slope should be shifted over such a width as to facilitate locating the slope and the crown of the directive dike under the bridge (Fig. 99).

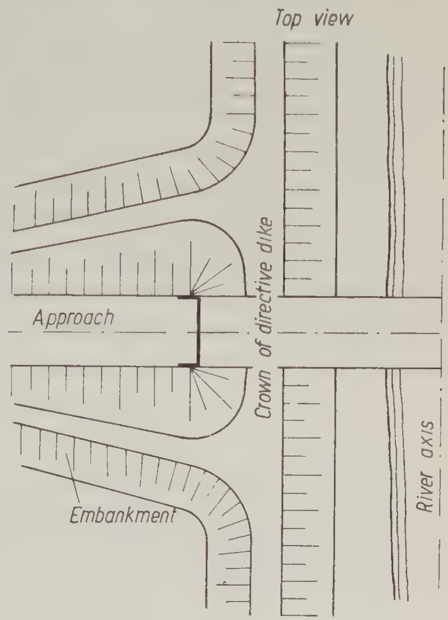


Fig. 99. Diagram of embankment approaches and directive dike

To facilitate the movement of vehicles from the road onto the crown of a directive dike, a suitably broad lift is built on the slope beginning at the level of the road and ending at the level of the crown of the directive dike.

There is no reason to fear a consequent decrease in the river cross section because, irrespective of the decrease, the conditions of discharge under the bridge may prove to be more favorable than in the case of an abutment protruding beyond the slope of a directive dike and causing considerable side contraction.

In the event of an excessive decrease in the river cross section under the bridge, the directive dike can be so located that the lower edge of its slope does not protrude beyond the wall of the abutment.

5. Revetting Banks and Bottom

Revetting of the slopes of directive dikes, approaches and cones of embankments should be undertaken within the limits of the bridge passageway.

In stable structures with openings smaller than 10 m, the bottom of a stream is sometimes revetted over the entire width and length of the opening. Generally, it is only a part of the stream bottom particularly exposed to scour near the directive dikes which is subject to revetting.

Choosing the type of slope revetment, the condition of its individual sectors should be taken into account. In this respect, the slope is divided into the following three parts:

(1) part I is situated below the level of the summer low waters; in this area, the slope is always flooded and, therefore, the revetting of this part of the slope comes within works underwater;

(2) part II is situated above the level of the summer low waters and reaches the level of the highwater; in this area, the slope is periodically flooded during the fluctuations of the water surface and, therefore, the work of revetting this part may be carried out in dry conditions during the summer low water stages;

(3) part III is situated above the level of highwaters; revetting the slope is necessary as a protection against the influence of precipitation.

Revetting Water Directive Dikes

The outer slope of the directive dikes (Fig. 100) remain under the influence of the action of slowly flowing water and, therefore, it does not require much revetting. A turf fixed with wooden pegs constitutes quite a sufficient revetment.

The inner slope of the directive dike is exposed to the danger of destruction by water streams flowing rapidly towards the opening. This slope should, therefore, be protected by a type of revetment which cannot be scoured.

The type of revetment is selected in correlation with the existing water velocity. Attention should be paid to the possibility of using local building materials. The velocity of the water current valid for computing a directive dike is equal to the mean velocity in a bridge opening.

The slope of the directive dike is revetted over its entire height, while the crown of the dike is not usually revetted.

Ground scour threatening to destroy a part of the bottom which constitutes a support for the revetment may appear along the base of a slope on the side of the main channel. A strong revetment is, therefore, placed along the base of the directive dike to protect the stream bottom against scour, and to constitute a foundation for the layer revetting the slope.

The head of the directive dike, near which the currents inflowing from various directions collide, is particularly exposed to scour.

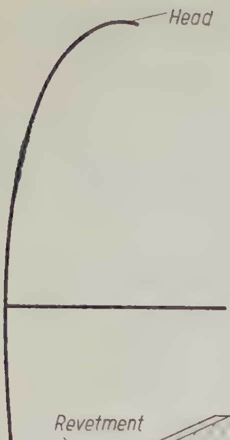
The unfavorable conditions in which the dike head works necessitate increasing its cross section and applying a more durable revetment.

For this purpose, the width of the head is increased to 3-4 m and the incli-

nation of its slopes usually has the value of 1 : 3 (Fig. 101). In the event of the appearance of a considerable scour, the inclination of the slope may reach 1 : 5.

Part II of the slopes requires revetting and sometimes also part I must be revetted if the directing wall crosses terrain depressions, bayous, lakes, etc.

Part I of the slopes is revetted with a rock filling layer of brushwood, or mattresses made of brushwood. A rock filling is spread on top of a thin



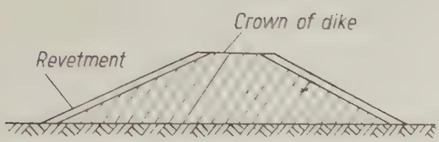


Fig. 100. Diagram of a directive dike

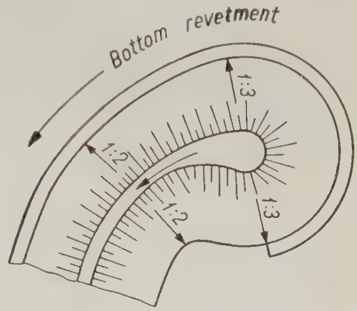


Fig. 101. Head of a directive dike

mattress made of brushwood if there is a possibility of the bottom settling — for instance if the bottom consists of peat — and, consequently, of the rock filling sinking deep.

Part II of the slopes is revetted with turf and single or double paving on moss, gravel or crushed stone. A dry stone wall and — for high velocities — gabions filled with stones are also used for this purpose.

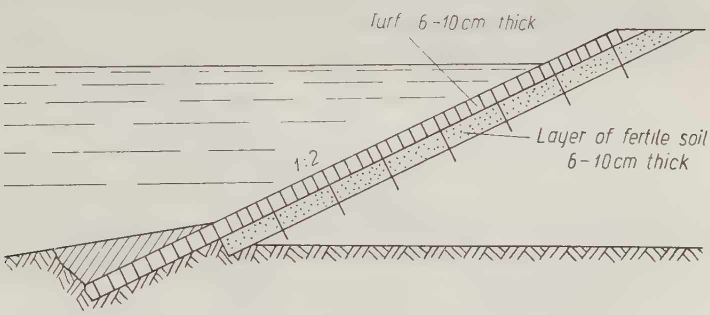


Fig. 102. Turf revetment of a slope

Note, that turf may be used for revetting banks only where a stream has low velocities. Turfs may be laid flat (grass upmost) or perpendicularly. Individual pieces of turf of sizes $0.5 \times 0.25 \times 0.08$ m are fixed to the slope with four

pegs 0.25—0.30 m long. A 10 cm thick layer of good soil is put underneath the turf.

The slopes should be revetted with turf during the wet season. Turf should be cut from meadows, in places where the grass is thick but short. The turf from marshes or very thin turf from meadows is not suitable for revetting directive dikes.

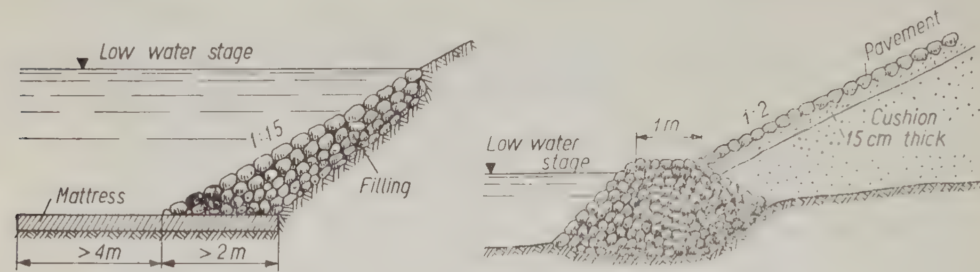


Fig. 103. Riprap

The rock filling is made in several layers (Fig. 103) and spread on the river bottom direct or on brushwood mattresses, wire nets, etc. Granite, sandstone

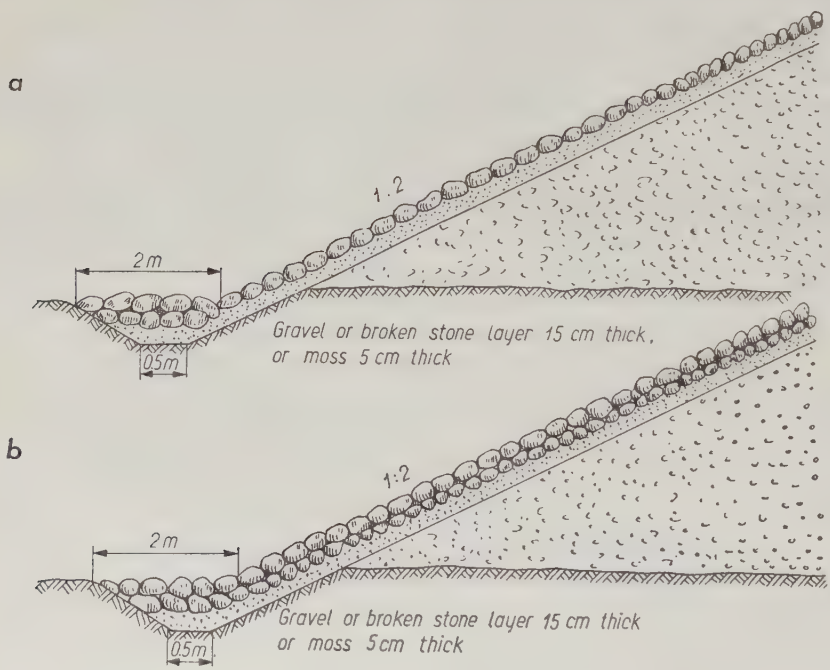


Fig. 104. Slope pavement, *a* — single, *b* — double

and compact limestone are among the generally used kind of stones, since they resist the action of frost and moisture.

Paving may be single or double (Fig. 104). Single paving is put on a layer of moss 5 cm thick and, if conditions are less favorable, on a layer of crushed stone or gravel 15-20 cm thick.

In the event of high current velocities, a double paving is applied, the upper layer consisting of larger stones, the lower of smaller.

Brushwood or brushwood mattresses may also be used for revetting part I of the slopes (Fig. 105). The application of these materials to revet part II of slopes should be considered as a temporary means, because such materials are liable to be destroyed when alternately exposed to the action of water and air.

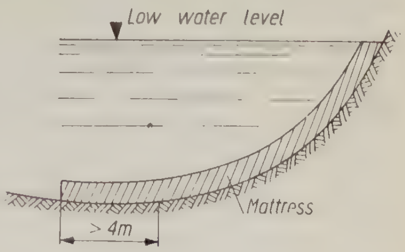


Fig. 105. Mattress revetment of slopes

For some velocities, the mattresses can be placed along the bank at intervals and not directly next one to another.

It is advisable to revet the scoured banks with gabions filled with stones (Fig. 106). In the event of bank scour and sinking of gabions, such a revetment is not liable to be destroyed and can be deformed only if its individual elements are insufficiently rigid.

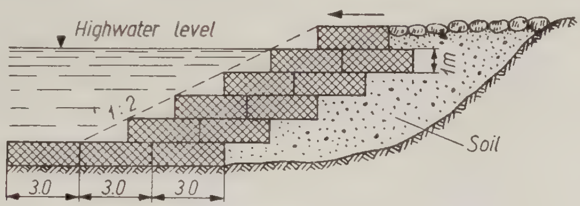


Fig. 106. Slope revetment by means of gabions

The revetting of a directive dike slope under mountainous conditions is shown in Fig. 107. Besides those already described, there are several other types

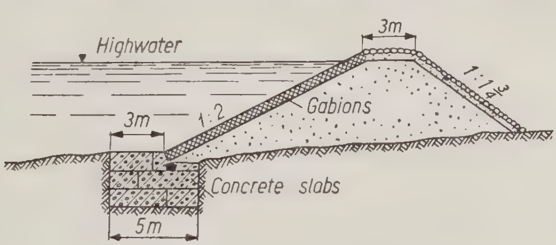


Fig. 107. Revetment of the slope of a directive dike in mountainous conditions

of revetments. Some of them are very expensive (e. g. concrete slabs) and, therefore, rarely used in practice.

Every type of revetment of slopes involves some admissible maximum velocity which does not cause scour and whose magnitude may be best determined by laboratory tests. The approximate values of admissible mean velocities in natural and revetted channels at an average water depth of 1 m are presented in Tables 77, 78 and 79.

For depth t larger than 1 m, the admissible mean velocity may be computed by multiplying the corresponding velocity shown in these tables by the depth t raised to the power $\frac{1}{3}$. The power $\frac{1}{5}$ should be applied in more important cases.

Table 77

Admissible Mean Velocities in Natural Channels at 1 m Depth of Water

No.	Soil characteristics	Soil particle diameter in mm	Admissible mean velocity in m/sec
1	Loose loam	0.005	0.25
2	Compact loam	0.05	0.35
3	Fine sand	0.25	0.50
4	Medium sand	1.0	0.60
5	Coarse sand	2.5	0.75
6	Fine gravel	5.0	0.90
7	Medium gravel	10.0	1.10
8	Coarse gravel	15.0	1.20
9	Small boulders	25.0	1.35
10	Medium boulders	40.0	1.60
11	Large boulders	75.0	1.90
12	Small stones	100	2.20
13	Medium stones	150	2.50
14	Large stones	200	2.80

Revetting Bridge Approach Slopes

The upper part of approach slopes located above the level of highwater is mostly protected against the action of precipitation by sowing grass or by a turf revetment.

A turf revetment laid in the form of bands, with free spaces left between the squares thus formed is applied to sandy soils (Fig. 108). To sow grass, on the other hand, is quite sufficient on other grounds.

The second part of the slopes of approaches situated above the level of the summer low water, and reaching the level of highwater, is revetted in a manner identical with that on the slopes of the directive dikes.

Table 78

Admissible Mean Velocities in Natural Channels Built of Cohesive Soils and Rocks at 1 m Depth of Water

No.	Soil characteristics	Soil cohesion	Admissible mean velocity m/sec
1	Rich clay (fractions smaller than 0.005 mm constitute over 50 percent of the weight of the sample)	low average high	0.4 0.6 0.9
2	Light clay grounds — (clay fractions constitute 12-18 percent of the weight of the sample)	low average high	0.6 0.8 0.9
3	Average clay grounds (clay fractions constitute 18-25 percent of the weight of the sample)	low average high	0.8 0.8 0.9
4	Common clays (clay fractions constitute 30-50 percent of the weight of the sample)	low average high	0.8 0.8 0.9
5	Heavy clay grounds (clay fractions constitute 25-33 percent of the weight of the sample)	low average high	0.7 1.0 1.2
6	Sandy clays (clay fractions constitute over 30 percent, the residue -dusty parts)	low average high	0.7 1.0 1.2
7	Loess	low average high	0.6 0.8 1.0
8	Compact turf		1.8
9	Peat on compact soil: (a) fine-fibered (b) coarse-fibered		 0.7 1.0
10	Marls, hardpans Dolomites, porous limestone, poor sandstone Layered limestone, hard sandstone Granites, basalts, diabases, andesites		2.2 3.0 4.5 7.0

Table 79

Admissible Mean Velocities in Revetted Channels at 1 m Depth of Water

No.	Type of revetment	Admissible mean velocity in m/sec
1	Turf placed flat on slightly cohesive ground	1.0
2	Turf placed flat on cohesive ground	1.2
3	Turf slabs placed perpendicularly	1.8
4	Rock filling	2.5
5	Fenced rock filling:	
	(a) diameter of stone 15 cm	3.0
	(b) diameter of stone 25 cm	3.5
6	Single paving:	
	(a) of boulder 15 cm in diameter	2.5
	(b) of boulder 20 cm in diameter	3.0
	(c) of boulder 25 cm in diameter	3.5
	(d) of 15 cm broken stone	3.0
	(e) of 20 cm broken stone	3.5
	(f) of 25 cm broken stone	4.0
7	Double paving:	
	lower layer — 15 cm stones, upper layer — 20 cm stones	4.0
8	Double paving bound with cement mortar	6.0
7	Brushwood or fascine bedding:	
	(a) 10 cm thick	1.4
	(b) 25 cm thick	2.0
10	Brushwood or fascine bedding loaded with stones	2.5
11	Brushwood or fascine mattresses:	
	(a) 30 cm thick	2.0
	(b) 50 cm thick	3.0
12	Brushwood of fascine mattresses loaded with stones	3.5
13	Gabions:	
	(a) 30 cm high	4.0
	(b) 50 cm high	5.0
14	Revetting mortared wall:	
	(a) of poor stones	5.0
	(b) of strong stones	7.0
15	Concrete revetting wall depending on the type of cement	6.0—8.0
16	Cribwork revetment	6.0
17	Wooden flume placed on firm ground with water flowing along the wood grains	8.0

The slopes of the approach embankments on the side of the upper water (above the bridge) are revetted according to the anticipated current velocity. The slopes of approaches on the side of the lower water are usually covered with turf, because the water velocity in the vicinity of an embankment below the bridge is slight.

The lowest part of the approach slopes located below the level of the summer low water is revetted with a rock filling.

If valley flats are wide, the water current velocities along the slopes of bridge approaches may have considerable magnitudes, and the cost of revetting such slopes is correspondingly high. For this reason cross dikes to divert water streams from the embankment (Fig. 109) are sometimes applied to protect the slopes of approaches. The cross dikes make costly revetting of slopes unnecessary.

The cross dikes are inclined against the current in valley flats at an angle of 75-80° in relation to the upper slope of the embankment. The heads of cross dikes are located along the line which connects the head of the directive dike with the edges of a valley flat at the point where such line is bisected by the approach embankment. The distance adopted as between cross dikes fluctuates between 2 to 4 times the lengths of the dikes.

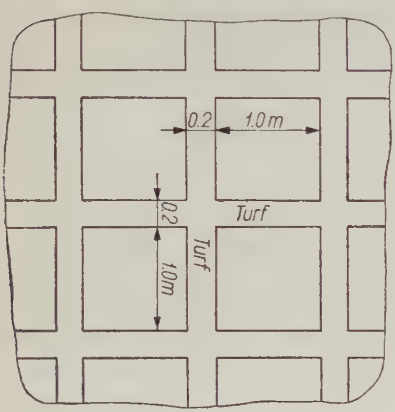


Fig. 108. Turf revetment of the sandy slopes of bridge approaches

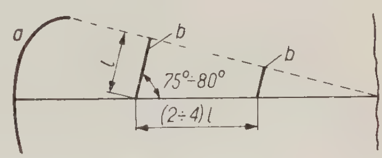


Fig. 109. Protection of approach slope
a — directive dike *b* — transverse dikes

Cross dikes are built of earth. Their crowns are 1-2 m wide and slope inclination 1 : 2. The slopes of cross dikes are revetted with paving or fenced rock filling.

Cross dikes should be so built that they will be partially flooded at the highest water level. The crown of a partially flooded cross dike is revetted in a manner identical with that of its slopes.

On mountain rivers, where depths are small and water current velocities great, unflooded cross dikes are designed, and have to be more massive than on the lowland rivers.

The slopes of the approach embankments are exposed to the destructive action of waves. Bushes and trees are planted along the base and on the slopes of embankments to protect approaches against wave movement. When floods are particularly deep, the slopes are protected by special floating devices.

As a result of the movement of waves, the water level near the slopes of approaches is higher than that of still water. For this reason the height of a wave above the level of still water should be taken into consideration when deciding the height of the crown edge or the upper edge of the revetment (Fig. 110).

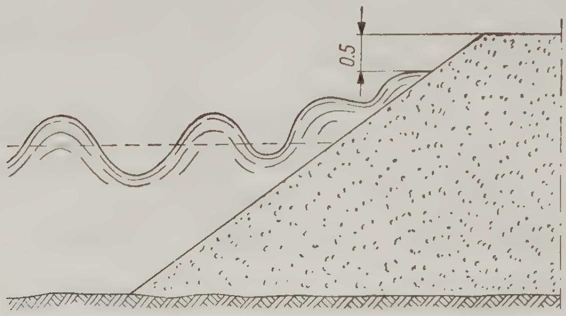


Fig. 110. Waves near an embankment

The size of a wave may be characterized by its length and height.

The height of a wave $2h$ is counted from the lowest point of water level during wave movement to the highest point of the crest of the flood wave; in other words, the height of a wave is equal to the entire amplitude of the fluctuations of the water surface.

The crest of the wave is elevated above the level of still water to the height h_g, slightly larger than half the full height of a wave — i. e. $h_g > h$. The length of the wave is equal to the distance between the uniform points of two waves.

The height and length of a wave depend on the velocity of wind and the length of the impetus of a wave. Many formulas have been elaborated for computing the elements of a wave; if H denotes the depth of a natural water reservoir, the following Andreev formula is used at $\lambda \leqslant 2H$

$$2h = 0.0208\, W^{5/4} D^{1/3}$$

$$\lambda = 0.304\, W D^{1/2}$$

where:

$2h$ — total height of wave, in m,
λ — length of a wave, in m,
W — wind velocity, in m/sec,
D — length of the impetus of a wave, in km.

If $\lambda > 2H$, $2H$ is adopted instead of λ in the formula for the wave length, and an approximate length of the impetus of the wave D is thus computed. The full wave height $2h$ is determined after computing D.

To use these formulas, the depth in valley flats H, wind velocity W — and its direction — as well as the length D of the impetus of the wave towards an embankment should be computed. $W = 15$—20 m/sec is adopted if the wind velocity is unknown.

The elevation of a wave in m, above the surface of still water during a progressive surface wave toward the slope of an embankment may be computed from the following formula derived by Diunkovskii:

$$h_f = 3.2\, K \operatorname{tg} \alpha \cdot 2h$$

where:

K — coefficient of roughness, and:

- K — 1.0 for a smooth slope i. e. concrete
- K — 0.9 for a slope paved or covered with turf
- K — 0.77 for a slope made of rock filling or overgrown by bushes and trees

α — angle of inclination of the slope plane to the horizontal,

$2h$ — total height of wave, in m.

Adopting the value $W = 17$ m/sec (generally occurring in practice), $tg\ \alpha = 1:2$, and slope revetting by paving or turfing for which $K = 0.9$, the following results are obtained:

$$\lambda = 5.17 \sqrt{D}$$

$$H_{\min} = 2.59 \sqrt{D}$$

$$2h = 0.72 \sqrt[3]{D}$$

$$h_f = 1.04 \sqrt[3]{D}$$

CHAPTER VIII

COMPUTING OPENINGS OF LARGE BRIDGES

The opening of a bridge l is the distance between the front walls of abutments measured on the water level accepted for determining the size of the opening. The same distance, minus the total widths of all the bridge piers, is called the clear span of a bridge l_1 (Fig. 111).

If a high water is contained under the bridge in an artificial or natural channel, the opening or span can be similarly computed between the slopes of the channel (Fig. 112):

$$l = b_1 + b_2 + b_3 + a_1 + a_2$$
$$l_1 = b_1 + b_2 + b_3$$
$$l_1 = l - a_1 - a_2$$

where:

b_1, b_2, b_3 — the clear span of individual bridge spans,

a_1, a_2 — widths of piers.

The length of a bridge L is the distance between the back walls of the abutments or, in bridges having a wooden roadway, the length of such roadway along the axis of the bridge (Figs. 111, 112).

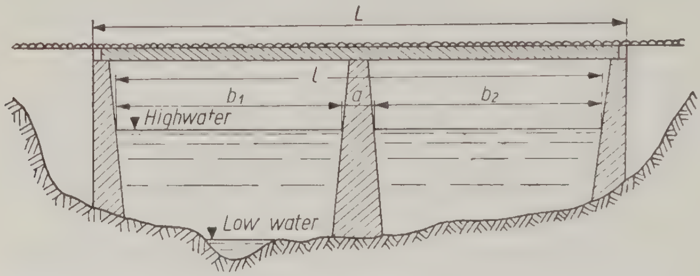


Fig. 111. Scheme of a reinforced concrete bridge:
L — length, l — opening, $b_1 + b_2$ — clear span of bridge

Computation of bridge openings consists in:

(a) determining the smallest cross section under the bridge necessary to pass the rise with an adopted frequency; the area of the cross section filled with flowing water is called the discharge section area or the active cross section;

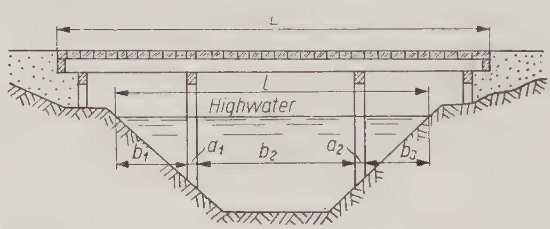


Fig. 112. Scheme of a wooden bridge:
L — length, l — opening, $b_1 + b_2 + b_3$ — clear span of bridge

(b) finding the appropriate value of the size of the opening in correlation to the existing cross section of the chanel within the limits of a bridge passageway.

The magnitude l of the bridge opening can be determined on the assump-

tion that the discharge section area will not be contracted. This condition holds if the bridge opening is equal to or larger than the width of a valley flat valid for computing the discharge. No headwater will in this event occur; therefore no directive dikes are built there, neither is any river training work performed.

Although the cost of building a bridge longer than the width of a valley flat is usually high, such a solution is at times advisable and even necessary. Mountain rivers marked by the high velocities of current and having variable shifting channels are often crossed without contraction of the cross section.

If the ground in valley flats is unsuitable as a support for the bases of an embankment — for instance if it consists of marshy peats and loams — a bridge instead of embankments should be built in valley flats, because to stabilize the embankments would involve the investment of large sums.

If there is a necessity to build a river crossing in a hurry (e. g. during war time), then, too, bridges are built instead of high embankments. They can be constructed much more quickly than high embankments, especially since timber may be used for constructing them.

To build a bridge in valley flats may sometimes prove cheaper than the construction of an embankment — for instance, if the road is high and no materials for embankments are available in the vicinity.

Usually, it is less costly to build an embankment than a bridge. For this reason, bridges shorter than the width of valley flats are usually designed and earth embankments are built at both ends.

A contraction of the discharge section area appears in such cases under the bridge and the effective volume of water flows through the opening at greater velocities than in a free channel. The elevation of water above the structure appears simultaneously.

In the event of a bridge smaller than the width of a valley flat being built, the following elements should be determined by computation:

(a) required discharge section area under the bridge,
(b) size of the bridge opening,
(c) maximum surface velocity, and mean velocity under the entire bridge and its individual spans,
(d) water elevation above the structure.

1. Determining the Necessary Area of Discharge Under the Bridge

If a bridge smaller than the width of a valley flat is built on an untrained stream, water below the bridge does not flow through the entire cross section of the stream but only through the central part, the eddies appearing near the banks.

At some distance below the bridge, the width of the cross section through which water flows will be smaller than under the bridge and, therefore, it is

the cross section which should be adopted as valid for computing a bridge opening.

The use of directive dikes decreases the scour and contributes to the extension of the contracted cross section below the bridge; therefore, such dikes should be built in any case. Bearing this fact in mind, we shall examine a case in which the directive dikes are constructed and the most contracted cross section within the limits of a bridge passageway appears under the bridge.

In the absence of directive dikes, the cross section under the bridge may also prove to be the smallest within the bridge passageway if the contraction of a stream below the bridge is small and the bridge cross section is considerably contracted consequent upon the construction of several pillars in the channel.

Building directive dikes involves, of course, a certain cost but it enables the bridge opening to be diminished, since the necessity to contract the stream below the bridge is avoided and the costs of maintenance of the bridge passageway are reduced in view of the considerably decreased deformations of the channel of the stream.

The smallest discharge section area under the bridge can be determined by one of four methods based on:

(a) discharge and average velocity in the main channel;
(b) hydraulic equivalents;
(c) morphological characteristics;
(d) coefficients or natural areas.

Establishing the Discharge Section Area under the Bridge on the Basis of Discharge and Average Velocity in the Main Channel

Using this method, the smallest discharge section area under the bridge is computed from the formula:

$$P = \frac{Q}{\mu\, v_o}$$

where:

P — smallest discharge section area under the bridge after subtracting the area of pillars, in sq m;

Q — discharge with adopted probability of occurrence for the entire cross section (of channel and valley flats), in cu m/sec;

v_o — average velocity in m/sec in the main channel before the building of a bridge for a discharge with adopted probability;

μ — coefficient of narrowing.

If the bridge opening is located at an angle α to the direction of the stream at a water stage adopted in computation, the discharge section area P under the bridge should be increased by dividing it by cos α. If a bridge is situated

obliquely to the direction of the current in the main channel, this correction should be introduced only for that part of the discharge section area which is contained within the limits of the main channel.

Water does not flow under the bridge through the full cross section, because a part of this cross section is occupied by the bridge piers and another part, near the pillars works less intensively, the eddies formed in these areas being caused by insufficiently streamlined shapes of piers and the oblique relationship of separate parts of the stream to the pillars. For this reason, the discharge section area decreases under the bridge, a factor to be taken into account in computation by means of a coefficient of contraction.

In the case of streamlined piers and fully perpendicular streams in the bridge cross section, the value of the coefficient was determined on the basis of studies and laboratory investigations.

Table 80

Values of Coefficient of Contraction μ

No.	Conditions of discharge	Lengths of bridge spans in m									
		4	6	10	20	30	50	80	100	150	200
1	Ice drift occurs at the water stage adopted for computing the size of an opening	0.70	0.80	0.85	0.90	0.95	0.97	0.98	0.99	0.99	1.0
2	Ice drift occurs under normal conditions	0.80	0.85	0.90	0.95	0.96	0.97	0.98	0.99	0.99	1.0
3	Single-span bridge with streamlined abutments and directive dikes	0.95	0.95	0.96	0.97	0.98	1.0	1.0	1.0	1.0	1.0

Table 80 presents the value of the coefficient of contraction μ determined by Boldakov. In his computations, Boldakov assumed that the eddies are formed on a width of 0.5 m around the pier and diminish the velocity of discharge. In addition, Boldakov assumed that the water streams flow obliquely to the pier at an angle of 5°.

A mean value of coefficient of contraction is adopted if there is a small difference in lengths as between particular bridge spans. Different coefficients are adopted for single spans if the ratio of span lengths is greater than 1 : 2.

Formulas for computing the coefficient of contraction in correlation to the shape of pillars and current velocity have been derived by Ozherelyev on the basis of observations of discharges of high waters under bridges.

For pillars having the front part shaped as an elongated triangle, Ozherelyev presents the following formula for computing the coefficient of contraction:

$$\mu = \frac{L - 2\,(0.625\,v - 0.20)\,(n - 1)}{L}$$

For the pillars having front parts semi-circularly shaped, this formula takes the following form:

$$\mu = \frac{L - 2\,(0.714\,v - 0.18)\,(n - 1)}{L}$$

where:

L — numerical value of the distance between abutments reduced by the width of piers, in m,

n — number of bridge pillars,

v — numerical value of the water velocity adopted for computing the bridge, in m/sec.

The contraction of the cross section under the bridge consequent upon ice drifts has not been taken into account by Ozherelyev in his derivation of formulas for determining the coefficients μ, because in most cases the crest of the flood wave appears on large and average rivers after the culmination in the ice drift.

The computation of coefficients μ by the Ozherelyev formulas is recommended for rivers with slight ice drift. On the other hand, in designing bridges on rivers where the peak of the flood wave occurs during an intensive ice drift, the coefficient should be adopted according to the Table presented by Boldakov (Table 80).

Establishing the Discharge Section Area under the Bridge on the Basis of Hydraulic Equivalents

For determining the discharge section area under large bridges, a method not requiring the direct computation of discharge, mean velocity and slopes,

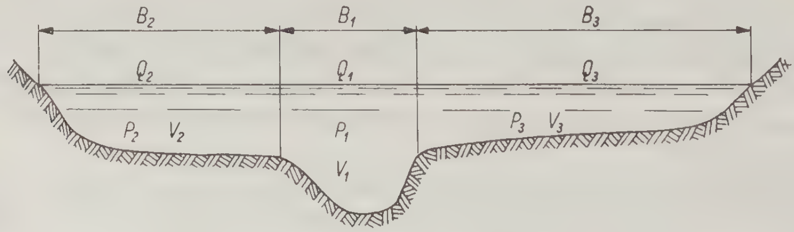


Fig. 113. River cross section divided into three parts

was published in 1914 by Kanshin. Using this formula, the discharge section area under the bridge may be computed without observations; the results ob-

tained are very approximate and, therefore, suitable for preliminary computations only.

Let us assume that the entire discharge section area, depending on the coefficient of resistance of water movement, was divided into characteristic sectors with areas P_1, P_2, P_3 and velocities v_1, v_2, v_3 (Fig. 113).

Then, the discharge before the bridge was built, amounts to:

$$Q = Q_1 + Q_2 + Q_3 = P_1 v_1 + P_2 v_2 + P_3 v_3$$

where:

P_1 — area of a cross section of the main channel;
P_2, P_3 — areas of cross sections in valley flats;
v_1 — velocity in the main channel;
v_2, v_3 — velocities in valley flats.

After a bridge has been built, the discharge amounts to:

$$Q = \mu P v$$

where:

P — area of the discharge section area under the bridge;
v — velocity under the bridge;
μ — coefficient of contraction.

These equations give:

$$\mu P v = P_1 v_1 + P_2 v_2 + P_3 v_3$$

or

$$P = \frac{1}{\mu v} (P_1 v_1 + P_2 v_2 + P_3 v_3)$$

Determining

$$\frac{v_2}{v_1} = K_2 \text{ and } \frac{v_3}{v_1} = K_3$$

and assuming after Kanshin that the velocity v under the bridge cannot vary from the existing velocity in the main part of a free channel v_1, the following magnitude of the discharge section area under the bridge is arrived at:

$$P = \frac{P_1 + K_2 P_2 + K_3 P_3}{\mu}$$

The ratios of velocities in valley flats to the velocities in the main channel K_2 and K_3 are called the hydraulic equivalents.

Given great width of the channel, it was assumed by Kanshin, for computing hydraulic equivalents, that hydraulic radii R are equal to the mean depths H.

The Chezy-Bazin numerical formula is recommended by Kanshin for computing the values K_2 and K_3. After algebraic transformations, the following correlation is obtained when slopes in a channel are equal to the slopes in valley flats.

$$K_2 = \frac{\gamma_1 + \sqrt{H_1}}{\gamma_2 + \sqrt{H_2}} \cdot \frac{H_2}{H_1}$$

$$K_3 = \frac{\gamma_1 + \sqrt{H_1}}{\gamma_3 + \sqrt{H_3}} \cdot \frac{H_3}{H_1}$$

where:

γ_1 — coefficient of roughness in the main channel = 0.85;

γ_2, γ_3 — coefficients of roughness in the valley flats = 1.75.

Kanshin maintains that knowledge of the magnitude of discharge and velocity of water is not necessary to determine the discharge section area under the bridge. Since the velocity under the bridge does not change, and is equal to that in a free channel, the discharge has nothing in common with the clear span of a bridge.

Kanshin's standpoint certainly deserves attention, because if there was a possibility of determining an accurate value of coefficients K_2 and K_3, the discharge section area under the bridge necessary for passing a valid discharge would be simultaneously and accurately determined.

Establishing the Discharge Section Area under the Bridge on the Basis of Morphological Characteristics

According to the state of knowledge of that time constant coefficients of roughness $\gamma_1 = 0.85$ for the main channel, and $\gamma_2 = \gamma_3 = 1.75$ for the valley flats were adopted by Kanshin.

The method described above for computing a required discharge section area under the bridge was developed by Professor Sribny, who applied the variable coefficients of roughness. The Sribny method for determining the discharge section area under the bridge is called the method of morphological characteristics. Sribny introduced a coefficient m, characterizing the change of conditions of discharge on a river:

$$m = \frac{v_1}{v}$$

where:

v_1 — velocities of water before building a bridge,

v — velocities of water after a bridge is built.

The value of this coefficient depends on the depth of scour and amounts to 0.85 for clay, 0.90 for sand, and 0.95 for loam.

The following formula was used by Sribny for computing hydraulic equivalents:

$$K_2 = \frac{v_2}{v_1} = \frac{n_1 R_2^{2/3} i^{1/2}}{n_2 R_1^{2/3} i^{1/2}} = \frac{n_1 R_2^{2/3}}{n_2 R_1^{2/3}}$$

where:

n_1, n_2 — coefficients of roughness in the main channel and the valley flat (Table 19);

R_1, R_2 — hydraulic radii in the main channel and the valley flat,

i — slope of water surface in the main channel and the valley flat.

As a result, Sribny derived the following general formula for computing the discharge section area under the bridge:

$$P = \frac{m}{\mu}\left[P_1 + \sum_{1}^{n} \frac{n_1}{n_n}\left(\frac{H_n}{H_1}\right)^{2/3} P_n\right]$$

where:

m — coefficient of the characteristics of the change in discharge;

μ — coefficient of contraction;

P_1 — discharge section area in the main channel;

n_1 — coefficient of roughness in the main channel, which should be adopted as in the Table elaborated by Sribny (Table 19);

H_1 — depth in the main channel;

H_n — depths in the individual sectors of valley flats;

n_n — coefficients of roughness in individual sectors of the valley flats;

P_n — discharge section areas in individual sectors of valley flats.

Hydraulic equivalents and morphological characteristics facilitate speedy determination of a discharge section area under the bridge without necessitating field observations. They do not, however, connect the sizes of openings obtained with discharges or stages with defined probability of occurrence. These methods can be used only when the probability of occurrence of the highest water level determined is identical with the probability adopted for computing the opening.

Establishing the Discharge Section Area under a Bridge on the Basis of Coefficients or Natural Areas

This method, worked out by Boldakov, consists in studying the discharge section areas under already existing bridges and in using these areas in designing a new bridge passageway.

Among all methods for determining the discharge section area under a bridge, this method, based on the natural areas, may be considered the most rational since it is not a theoretical area which is obtained but a natural area nearest to the magnitude necessary for the water discharge.

This method can be used only when other bridges are located in the neighborhood of the building site of the intended bridge.

The method of computing the discharge section area under the bridge varies according to the object of computation, i.e.:

(1) designing a new bridge alongside an already existing one,
(2) rebuilding an existing bridge,
(3) designing a new bridge at some distance from an existing one.

Designing a New Bridge alongside the Existing One

Such cases generally occur on the railroad when a second track is laid. Using the method described, it is necessary to take the measurement of the cross section under the existing bridge, determine the level of high water and compute for this level the discharge section area P_2 under the bridge.

The discharge section area P_1 under the intended bridge is computed from the formula:

$$P_1 = \frac{\mu_1}{\mu_2} P_2$$

where:

P_1 — discharge section area under the intended bridge,
P_2 — discharge section area under the existing bridge,
μ_1 — coefficient of contraction under the intended bridge,
μ_2 — coefficient of contraction under the existing bridge.

Rebuilding an Existing Bridge

This usually eventuates from the necessity to replace an existing bridge on a piling by a bridge on stone or concrete piers.

Determining the discharge section area by the method presented above is difficult, because the openings of wooden bridges are usually computed without taking into account the scour. Further, in valley flats wooden bridges have additional spans instead of embankments. For this reason, the discharge section area of such bridges is too large and is not reliable for computing a new bridge.

In such cases the method of morphological characteristics should be applied (Fig. 114). After the coefficients of roughness for the main channel and valley flats have been determined, the required discharge section area P is found from the following formula:

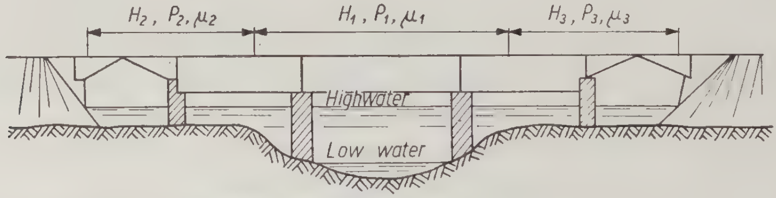


Fig. 114. Scheme of a wooden bridge for the computation of a discharge section area

$$P = P_1 \mu_1 + P_2 \mu_2 \frac{n_1 H_2^{2/3}}{n_2 H_1^{2/3}} + P_3 \mu_3 \frac{n_1 H_3^{2/3}}{n_3 H_1^{2/3}}$$

where:

P_1, P_2, P_3 — the areas of the main channel reduced by the widths of bridge piers and of the left- and right-hand parts under the bridge;

μ_1, μ_2, μ_3 — coefficients of contraction;

n_1, n_2, n_3 — coefficients of roughness.

Designing a New Bridge on the Same River at Some Distance from the Existing One

In this case, the dsicharge section area can be transferred from the already existing to the new bridge by means of the following formula:

$$P_1 = P_2 \frac{F_1^n B_1^m i_1^{1/4} G_2}{F_2^n B_2^m i_2^{1/4} G_1}$$

where:

P_1 — required discharge section area in sq m;

P_2 — discharge section area in the existing bridge passageway in sq m;

F_1 — area of catchment basin in sq km from river sources to the intended bridge;

F_2 — area of catchment basin in sq km from river sources to the existing bridge;

B_1 — width of catchment basin in km from river sources to the intended bridge, being equal to the quotient of the area of the catchment basin situated above the bridge divided by the length of this part of the catchment basin (not the length of the river);

B_2 — width of catchment basin in km from river sources to the existing bridge;

n, m — coefficients depending on the type of rise, taken from Table 53;

i_1 — slope of catchment basin from river sources to the intended bridge expressed as a decimal fraction;

i_2 — slope of catchment basin from river sources to the existing bridge expressed as a decimal fraction;

G_1 — geological coefficient of the ground within the limits of the intended bridge taken from Table 81;

G_2 — geological coefficient.of the ground within the limits of the existing bridge, taken from Table 81.

Using the method of natural areas, the discharge section area under the existing bridge should be reduced to the defined probability of a rise. The probability of occurrence under the existing bridge is computed for this purpose

Table 81

Geological Coefficients for Various Soils

Characteristics of soils		Admissible mean velocities under bridges in m/sec	Geological coefficient G
According to geological structure	According to scour resistance		
Loam, fine sand	Weak soil	1.3	0.75
Coarse sand or clay with loam layers	Medium soil	1.6	0.90
Coarse sand with boulders, clay	Medium soil	1.8	1.0
Gravel	Resistant soil	2.0	1.1
Boulders	Resistant soil	3.0	1.6
Large boulders	Resistant soil	4.0	2.2
Rock	Unscourable soil	10.0	—
Mixed soils	Weak or average soil overgrown with grass (outside the limits of low summer stages); the discharge section area of this part of the cross section under the bridge at the highwater stage amounts to:		
	20% of the entire discharge section area	1.8	1.0
	30% of the entire discharge section area	2.0	1.1
	40% of the entire discharge section area	2.3	1.3
Mixed soils	Medium soil scoured over a certain space up to the unscourable soil (within the limits of the low summer stages); discharge section area of this part of the cross section under the bridge at the highwater stage amounts to:		
	20% of the entire discharge section area	1.8	1.0
	30% of the entire discharge section area	2.0	1.1
	40% of the entire discharge section area	2.3	1.3

on the basis of the high water stage ascertained. If the probability thus determined varies from that adopted for computing a new bridge, the discharge and water stage should be correspondingly changed, which would also cause a change in the magnitude of the discharge section area.

The magnitude of the discharge section area under the bridge may fluctuate: it increases during the rise and, due to silting, decreases somewhat after the passage of high water.

Moreover, the discharge section area under the bridge may vary consequent upon continuous deformations of the surface of the valley flat. This may

change the gradient of the river bottom which, therefore, causes scour or silting of the channel.

The discharge section area under the bridge varies in proportion to the change in the shape of the channel. These circumstances should be taken into account in building the foundations of the piers particularly when wooden piles are used for this purpose.

It is clear from the above that the magnitude of the anticipated changes in the transferred discharge section area should, if possible, be taken into account. The largest area is adapted for computing the passageway of the intended bridge and a margin of 10 to 20 percent is added to allow for silting.

2. Elevation above the Bridge

After a bridge has been built an elevation arises following the contraction of the cross section of the stream. As a result of this contraction, the velocity v_o in a free channel is different from the velocity under the bridge v_1.

The magnitude of the elevation h can be computed from the following formula:

$$h = \lambda (v_1^2 - v_0^2)$$

where symbols denote the numerical values of the individual magnitudes:

h — elevation above the bridge, in m;

v_1 — mean velocity after or before the scour, in m/sec;

v_0 — mean velocity of the entire stream in m/sec, obtained by dividing the entire discharge by the entire area of the stream together with valley flats;

λ — coefficient, which according to Boldakov's studies should be adopted as being equal to 0.1.

The highest elevation appears at a certain distance above the bridge. For considerable contractions of very large rivers, this distance may even reach several kilometers.

The longitudinal cross section of the river near the bridge is shown in Fig. 115.

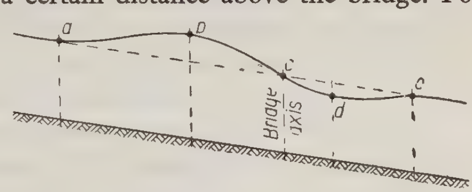


Fig. 115. Longitudinal cross section of a river near the bridge before the scour: *a* — end part of the elevation, *b* — greatest elevation, *c* — water level along the bridge axis, *d* — extreme fall of water level, *e* — end part of fall of a water surface

The velocity v_1 — appearing in the formula for computing elevation — is a mean velocity after scour. It may happen, however, that a rise with a magnitude adopted in computations occurs even during the first year after the bridge has been built. For this reason, some hydrologists recommend adopting in the

formulas for computing elevation the mean velocity before the appearance of the scour instead of the mean velocity occurring after the scour.

It very rarely happens, however, that the adopted maximum discharge occurs immediately upon the completion of a bridge. Therefore, the velocity after scour should be taken into account and only in very important cases need the velocity prior to scour be used in computations.

Coefficient λ for computing elevation not lower than 0.1 should be used, because such values were obtained by observation in practice.

Some hydrologists recommend determination of elevation from the following formula:

$$h = \frac{v_1^2 - v_o^2}{2g}$$

This formula is derived from the Bernoulli equation, disregarding losses and contraction of cross section. It yields a diminished magnitude of the elevation because the magnitude of the resistance increases together with the rise in velocity.

Such a formula cannot be used in computing elevation above the bridges, since it was derived on the basis of erroneous premises, and the more so since no investigator has been able to prove the correctness of the formula by way of field studies or laboratory research.

If elevation is to be computed directly under the bridge, the following numerical formula can be used:

$$h = 0.05\,(v_1^2 - v_o^2)$$

It has been established on the basis of observation, that:

(a) discharge always extends under the bridge, while water surface under the bridge slopes slightly towards the valley flat;

(b) in the absence of directive dikes, the elevation above the bridge increases about 1.5 times;

(c) the magnitude of elevation is only insignificantly diminished by the length of the directive dikes being doubled;

(d) velocity and slope of elevation above the embankment increases markedly in the vicinity of the bridge opening;

(e) the slope of the elevation surface below the bridge and along the embankment is insignificant, and can be assumed as a horizontal;

(f) elevation along the axis of the bridge (counting from the surface in the free channel) can be determined from the numerical formula:

$$h = 0.1\,(v_1^2 - v_o^2) + i\,l_1$$

where:

i — slope of river channel expressed as a decimal fraction;

l_1 — length of the river sector from the head of the upper directive dike to the abutment, in m

(g) below the bridge, a degradation of the water level appears in correlation to the surface in the free channel, which can be computed from the formula:

$$h = -i\,l_2$$

where:

l_2 — length of a river sector from the abutment to the end of the lower directive dike, in m.

3. Determining Admissible Velocity under a Bridge

It results from the formula for computing the discharge section area P, that this area is inversely proportional to the velocity. Consequently, the adoption of a greater velocity is desirable in order to obtain a shorter bridge if such does not cause any significant change in the existing conditions of discharge.

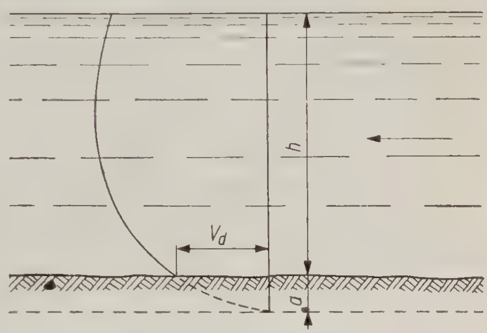


Fig. 116. Distribution of velocities in a vertical: h — stream depth, a — mobile bottom layer, v_d — bottom velocity of water

Velocities which do not cause scour are usually lower in the river than those existing in a free channel. For this reason, the channel is always scoured, and the layer of bottom material is shifted along its bed (Fig. 116).

The bottom velocity always has a certain extreme value, and the decrease in the velocity to zero appears in the ground below the level of the channel bottom. The thickness of the moving layer of the ground is undefined. Some investigators suppose that it amounts to about 1 m.

Although a velocity which does not cause bottom scour is usually smaller than that appearing in the stream, this does not mean that the magnitude of the admissible velocity should be less than the existing one. Such an assumption cannot yield satisfactory results.

If a velocity which does not cause scour of the channel ground is adopted as valid for computation, it will contribute to an artificial extension of the bridge opening. Such a computation might even seem to prove that a part of a bridge should be located in the unflooded area, which certainly could not diminish the velocity existing in a free channel.

For this reason, velocities which do not cause scour of the ground can be adopted only in computing such artificial channels as canals, ditches, etc. In

these channels, the scoured places cannot be filled up again since no sediment is transported from the upper reaches of the stream.

The velocity existing in the channel can best be determined by hydrometric measurements and adopted as valid for computing a bridge opening. Since the existing velocity is very seldom directly observed at high stages, extrapolation to the level adopted in computation is usually performed. Extrapolation should however be effected with extreme care, because the mean velocity of the main channel increases slowly at high water stages, since scour takes place at the same time.

If a considerable part of the water, say, 50 percent, flows within the limits of a valley flat, no increase in the mean velocity can occur. If more than 60 percent of a discharge moves along the valley flats, the mean velocity in a main channel may drop even during rising water stages. In this event, the highest velocity observed should be adopted as valid.

When no direct measurements are available, the mean velocity can be determined by empirical formulas, principally the Chezy type formulas. Computations made according to these formulas are, however, inaccurate. In determining discharge and mean velocity by empirical formulas, an error is committed in one respect. If the discharge is subsequently divided by the mean velocity, the error is diminished.

A different situation arises when various tables are used in which mean velocities existing in a channel are presented for various types of soil (Table 82, after Lishtvan). Different methods have been applied to elaborate these tables and, therefore, instead of reducing the errors, as in the previous case, they can be summed up. For this reason, the use of tables is permissible only in preliminary computations.

4. Discharge Section Area under the Bridge

After the discharge section area under the bridge has been established the size of a bridge opening is determined by way of geometrical computations. That is to say, a necessary discharge section area for the water stage adopted is selected in the cross section of the bridge.

The size of a bridge opening depends to a considerable extent on the shape of the cross section of the stream across which a bridge structure is intended. For identical area of the cross section, the size of an opening decreases as the mean depth of the stream increases.

Various shapes of the cross section of the stream with identical area P are shown in Fig. 117. From this figure it is seen that the deeper and wider the main channel, the smaller the length from which the required discharge section area can be chosen. The size of the bridge opening will, therefore, be smaller in such a cross section than in a cross section with a flat and poorly developed main channel.

Table 82

Mean Velocities in m/sec Existing in the Main Channel for Various Soils and Depths at Frequency 1 : 100

Soils	Characteristics of soils	Grain dimensions in mm	Mean depths in m										
			2	3	4	5	6	8	10	12	14	16	18
Sandy	Loamy fine-grained sands, light sandy -clay soils	0.005—0.25	0.70	0.80	0.90	1.00	1.10	1.20	1.30	1.40	1.50	1.55	1.60
	Average and variously grained sands, and fine-sands with gravel	0.25—5.0	0.90	1.00	1.15	1.30	1.40	1.55	1.70	1.80	1.90	1.95	—
	Various — and fine-grained sands with gravel and fine boulders	1.0—25.0	1.30	1.50	1.65	1.75	1.85	2.00	2.15	2.25	2.35	—	—
Boulders	Medium-sized boulders with sand and gravel	2.5—40.0	1.80	2.10	2.25	2.40	2.60	2.75	2.90	3.00	—	—	—
	Large boulders with gravel	15.0—75.0	2.50	2.80	3.10	3.30	3.50	3.70	3.85	—	—	—	—
	Large and very large boulders	40.0—100	3.20	3.60	4.00	4.40	4.70	5.30	—	—	—	—	—
	Large and very large boulders with rocks	75—200 and more	4.20	4.90	5.60	6.10	—	—	—	—	—	—	—
Clay	Weak loamy clays and clay-sandy soils, peats	—	0.80	1.05	1.20	1.35	1.45	1.65	1.80	1.90	2.00	—	—
	Clay-sandy soils and average compact clays — loess	—	1.15	1.40	1.60	1.70	1.80	2.00	2.10	2.20	—	—	—
	Compact clay, very compact clay -sandy soils (morene)	—	1.45	1.70	1.90	2.05	2.15	2.30	2.40	2.50	—	—	—

This is why in selecting the route of the passageway of the future bridge, a cross section of the stream should be chosen within the limits of which the main channel is wide and deep in order to diminish the size of the bridge opening.

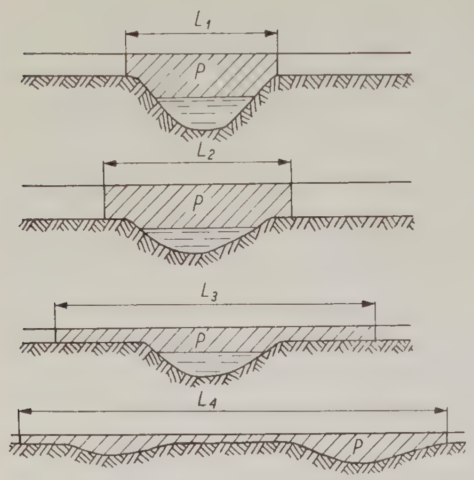


Fig. 117. Correlation between the size of a bridge opening and the channel configuration

The ground scour effected by water flowing under the bridge may sometimes diminish the size of the bridge opening. Such scour is taken into account in computing the necessary discharge section area by adopting the corresponding velocity under the bridge.

The scour may be taken into consideration only when a channel consists of scourable soils. All kinds of soil belong in principle to this type, except rocks and ground consisting of large stones and highly cohesive hardpans.

In poorly developed channels, the size of an opening sometimes happens to be large irrespective of the scour envisaged. In such cases, the discharge section area under the bridge is increased by artificial deepening of the bottom so as to diminish the size of the bridge opening, and to have a uniform distribution of velocities under all bridge spans.

Determining the Depth of Scour under the Bridge

Ground under the bridge can be scoured as a result of the action of the following factors:

(1) scour appears over the entire cross section under the bridge if in determining the bridge opening we adopt under the bridge a greater velocity than in a free channel; such scour terminates when, consequent on an increase in the discharge section area, the mean velocity under the bridge is balanced by the velocity in a free channel and the channel regains its dynamic balance;

(2) deepening of the main channel takes place when parts of the bridge cross section of a river, located in valley flats, are covered with turf, bushes or artificial revetment; in this case, the scour envisaged by computations does not appear in valley flats and the deepening of the main channel increases as compared with what was envisaged;

(3) the occurrence of deep scours in the central and extreme parts of a bridge opening may be caused by the absence of directive dikes within the limits of the bridge passageway;

(4) local changes in the shape of the channel under the bridge may occur in the vicinity of bridge pillars, because a transverse circulation of water is caused by piers.

Correct consideration of the scour near the bridge piers and over the entire width of the cross section is exceptionally important in designing bridges.

The scour of the bottom can be taken into account in computing bridge openings only if the main channel within the limits of the bridge passageway is straight over the length of at least double the width of the opening.

If the channel under the bridge follows a curvilinear direction, bottom scour may then be taken into account only in the central part of the channel and at the concave bank. Scour is not taken into account near the convex bank.

The question of admissible scour under the bridge should in every case be examined from the technical-economical point of view. It is necessary to ascertain which is the more favorable — shortening a bridge and deepening the pier foundations or lengthening the bridge on shallower foundations.

The degree of scour is determined by coefficient of scour P. This is the ratio of the discharge section area computed to be necessary under the bridge after scour to the discharge section area before scour.

Limited magnitudes of scour should be adopted in computations. It has been established by studying the discharge section area under existing bridges, that in practice coefficient P mostly fluctuates within limits of 1.2 and 1.4, and very seldom reaches 1.5.

Values of admissible scour when foundations are unstreamlined and semi-streamlined are shown in Table 83. In some cases, a coefficient of scour equal to 2 can be permitted for flat channels for a small depth of mean scour amounting to about 1 m.

The depth of the foundation of the bridge is established on the basis of the mean and maximum scour under the bridge.

The depth of a stream after scour, chosen at random at any point of its cross section, is computed from the formula:

$$h_r = P\,h$$

where:

h_r — depth of water after scour, in m;

h — depth of stream before scour at the water stage adopted in computations, in m;

P — coefficient of scour.

The line of the mean bottom scour (Fig. 118) is obtained by plotting the depths computed by means of this formula at each point of the cross section under the bridge from the valid water stage.

The area of a cross section under the bridge limited by the water surface

Admissible Magnitude of Scour

No.	Type of pier foundation	Unstreamlined foundation within limits of the scour		Semistreamlined foundation within limits of the scour	
		coef-ficient of scour	percent-age of scour	coef-ficient of scour	percent-age of scour
1	Deep foundation, caissons, sunk wells, metal and ferro-concrete poles driven deeply into the ground and appropriately worked, and also foundations on rocks	1.3	25	1.40	30
2	Foundation on wooden piles within double sheet piles	1.1	10	1.25	20
3	Shallow foundation on piles and directly on the ground within single sheet piles	1.0	0	1.1	10
4	Crib props and foundations without piles and sheet piles	1.0	0	1.0	0

at the water stage adopted in computations, and the line of the mean scour should be equal to the discharge section area under the bridge.

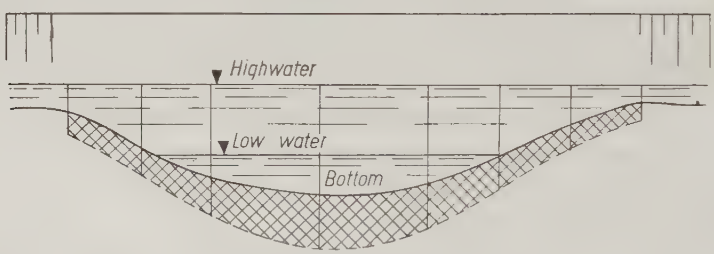


Fig. 118. Cross section of a stream under a bridge, with scour depth indicated

The line of the mean scour sometimes bisects the soils which are not subjected to scour. In this case, the line of the mean scour in the scourable part of the ground can be determined by the following Boldakov formula:

$$h_r = h\frac{PF_1 + (P-1)F_2}{F_1}$$

where:

F_1 — area of the cross section under the bridge at the valid water stage and in the part with scourable ground,

F_2 — area of the cross section under the bridge at the same water stage and in the part with unscourable ground,

h_r, h, P — as in former formulas.

The discharge section area limited by the line of the theoretical mean scour should be equal to the discharge section area under the bridge as computed. Uniform scour does not usually appear in a cross section, but maximum depths may occur at any point within the limits of the opening. Therefore, a line of maximum scour over the entire cross section should be drawn for determining the depth of pier foundations.

The line of maximum scour under the bridge is determined by the formula:

$$h_{max} = P\left[h + \frac{h}{H_m}(h_m - h)\right]$$

where:

h_m — possible depth of scour at a given point, in m;

H_m — maximum depth of water under the bridge prior to the scour;

P, h — as before.

This formula should also be used when $P = 0$ — i.e. when scour has not been taken into account in computing the opening. The increase in the depth of the bridge cross section of a river can be explained in this case by the irregular distribution of velocities on the width of the bridge cross section when there are no directive dikes within the limits of the bridge passageway.

The discharge section area under the bridge, with scour taken into account, is selected by means of the computed area divided by the coefficient of scour. When for instance, an area $F = 1{,}200$ sq m and coefficient of scour are equal to 1.2, the following area should be chosen:

$$1{,}200 : 1.2 = 1{,}000 \text{ sq m}$$

Artificial Deepening of the Channel under the Bridge

In addition to scour of the bottom by flowing water, an artificial deepening of the channel in sections situated above the level of low waters is sometimes applied to diminish the bridge opening.

Since a considerable amount of digging is involved in achieving the necessary depth under the bridge, the advantages and disadvantages of such a solution should be thoroughly studied and justified in each case. An economic point of view should be adopted, bearing in mind the cost of keeping the deepened sector in operation. If earthworks are too expensive, the size of the bridge opening should be increased.

The intended deepening of the channel can in sandy soils be only partially put into effect after removing the turf, grubbing bushes, and applying appropriately planned river training structures. The longitudinal ditches can then

be dug, appropriately connected with the general situation in a bridge passageway. Such a solution, however, can be applied only under particularly favorable conditions, because practice has shown that scouring of the ditches does not always occur; frequently, they become overgrown by vegetation and silted by sediment.

The deepening of a channel consists in the removal of the bank soil to the required depth in the bridge cross section of a river. The level to which ground can be deepened is taken to be about 50 cm above the surface of the summer waters; greater deepening is impossible in view of the great quantity of earthworks involved.

Artificial deepening should be conducted not only under the bridge but also above and below the bridge so that the streams may enter the bridge opening and, subsequently, emerge without any obstruction from it at a valid mean velocity. For this purpose, channels are also deepened above the bridge over a length usually equal to 0.50-1.0 of the length of the bridge and below the bridge over a length equal to 0.75-1.5 of the length of the bridge. If such works are not undertaken, the artificially deepened sectors do not work satisfactorily. The deepening causes a violent increase in the area of the cross section under the bridge and, consequently, a decrease in mean velocity. This creates favorable conditions for the deposit of suspended material such as is borne along by the river in great quantities during rises.

Bottom deepening under the bridge may be permitted only if:

(a) the main channel within the limits of the bridge passageway is straight over a length at least equal to double the width of an opening;

(b) deepening of the curvilinear sector is made on the concave bank; a deepening on the convex bank can be silted up due to the transverse circulation of water and therefore it is disregarded in the computations.

Steep banks built of strong soil offer the most favorable conditions for deepening. To attempt artifical deepening in low valley flats with weak, loose soils is hardly to be recommended.

Examples of a properly designed deepening of bottom are shown in Fig. 119.

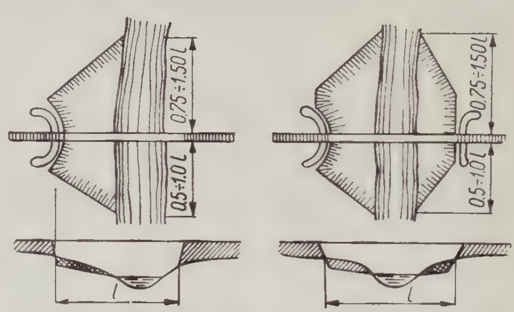


Fig. 119. Examples of artificial bottom scour under a bridge:
a one-sided artificial scour *b* two-sided artificial scour

The following conditions should be observed in bottom deepening if the intention is to achieve an actual increase in the discharge section area under the bridge and a uniform velocity of water flowing through such area:

(1) the deepened area should not exceed 20 percent of the entire discharge section area under the bridge;

(2) the limits and dimensions of the deepened sector should be suited to the directive dikes designed; the length of the deepened sector should be not smaller than that of directive dikes;

(3) the longitudinal slope of the deepened sector should be equal to the slope of the entire river; the depression dug should be connected with the natural surface of the ground by slopes inclined, above the bridge at 1 : 5, and below the bridge — at 1 : 10;

(4) the shape of the deepened sector should be formed according to the direction taken by the river during the flow of high waters;

(5) scour of the deepened sector is admissible, as in the case of the main channel; certain limitations should then be introduced, however, because of the danger of complete deformation of the river regimen; for this reason, the total areas involving scour and artificial deepening should not be greater on navigable rivers than 30 percent and on unnavigable rivers than 40 percent of the discharge section area required under the bridge.

5. Special Cases of Computing Bridge Openings

In addition to bridges with a single opening as generally built on rivers, there may be special conditions calling for a bridge passageway design based on different principles.

For instance, on some rivers with wide valley flats, the building of several separate bridges to pass the discharge may prove to be more advantageous. In such cases, besides the bridge opening in the main channel, one or more additional openings are also designed in deep parts of the valley flats.

It is then necessary to supplement the computations generally used in order to establish the amount of the discharge passing through the individual openings.

Special computations are also required for bridge passageways situated on a backflow of another, larger river, as well as for a river crossing where water is elevated by weirs located above or below the bridge.

Bridge Passageways on Rivers with Wide Valley Flats

Bridge passageways on rivers with wide valley flats effect considerable change in the regimen of the stream owing to the channel deformations they cause. Water from valley flats directed to the bridge, flows a long way along the embankments and may cause scour. In addition, passageways with wide valley flats and a poorly shaped main channel usually require, in order to pass the water, a good deal of artificial deepening under the bridge.

For these reasons, to build additional bridges in the deepest places of valley flats is a solution rational in the passageways where such areas are wide.

It may also prove to be necessary to design an additional opening when streams cross one another, in the place where a tributary having a common valley flat with the main river joins it, etc.

The construction of additional bridges in valley flats is not popular with designers, who fear that such bridges may be scoured during rises — a phenomenon actually observed in the field in some cases.

Particularly large scours appear when high velocities arise under a bridge in a valley flat due to a considerable difference of levels on either side of an embankment. Since the sediment is not moved by streams in valley flats, a channel under the bridge in such a flat is subject to considerable scour and the bottom often takes the shape of a trough.

Maximum quantities of water flowing through the main opening sometimes change direction, by changing their course to the additional opening in the valley flat. The result is that the bridge piers in the valley flat are scoured, and the bridge may collapse.

Such cases are, however, mostly caused by inadequate assumption of the quantity of water for each bridge. Consequently, a velocity lower than that computed appears under the main bridge and a much greater velocity under the additional bridge.

The intensity of deformation of the channel under an additional bridge is also increased as a result of the fact that such bridges, considered secondary, are usually built without directive dikes, which is inadmissible.

In a correct computation, the velocity under the bridge in a valley flat can be much lower than the velocity in the main channel.

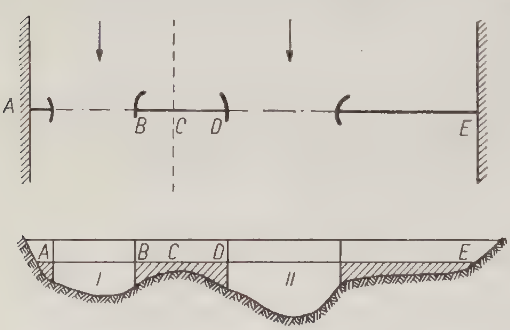


Fig. 120. Scheme for determining the division of discharge into two parts

If it is decided to locate an additional opening in a valley flat, the discharge which is to pass through each opening should be established.

In the present state of our knowledge, there are no accurate methods of determining the division of discharge in a principal and a supplementary opening and, therefore, approximate methods should be used.

Detailed studies should be conducted to find the line of division of a valley flat, and the division of discharge between two openings should be made accordingly.

In a case shown in Fig. 120, there is a probability that the highest point C would be an extremal point of division of the cross section of the stream.

It can be assumed, therefore, that the discharge in the valley flat between points A and C will approximately head for the left hand, and between points C and E — for the right hand opening.

It is assumed that an additional opening should pass at least 20 percent of the valid discharge. An additional opening should not be introduced if computations show that a smaller quantity of water would pass through the additional opening.

The following method of computing discharge in the channel and in valley flats is recommended by Boldakov:

(1) The size of an opening in the main channel and in a valley flat is assumed on the premise that the discharge section area under the bridge in a valley flat should pass at least 20 percent of the entire discharge.

(2) The quantity of water which passes the main channel and individual parts of the valley flat (these parts are denoted in Figure 121 by numbers 1, 2, 3, 4, 5) is determined in the field by way of hydrometric measurements. The cross section of the stream above the bridge passageway, where water movement conditions are not changed by building a bridge, is valid for taking measurements. The distance of this cross section from the bridge passageway is adopted as approximately equal to the width of a valley flat.

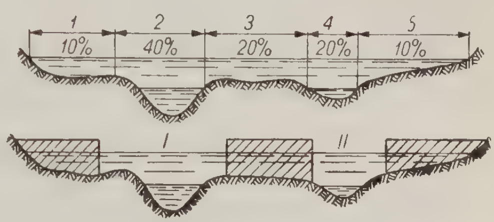


Fig. 121. Scheme of water discharge in the main channel and in valley flats

(3) The discharge is divided into parts, one of which will flow through the opening in main channel I and the other — in valley flat II. We may presume that the water from the left hand flat 1, from the main channel 2 and from the part of the valley flat 3 will reach the opening I. On the other hand, the water from the part of the valley flat in its central sector 3, from the depression in the valley flat 4 and from the valley flat near the right bank 5, will flow to opening II. The division of the discharge from the central part of the valley flat 3 as between the two bridges can be conducted proportionally to the discharges appropriate to each bridge.

Taking the coefficient of irregularity as being equal to 1.1, we obtain the following percentage of the discharge which should be passed through the opening I: $1.1\left(10 + 40 + 20\,\frac{40}{40+20}\right) = 69$ percent. Opening II in the valley flat can pass: $1.1\left(20\,\frac{20}{40+20} + 20 + 10\right) = 41$ percent of the existing discharge.

(4) The velocity is checked under bridge II. This velocity determined by hydrometric measurements should be not greater than the velocity admissible for weak grounds in valley flats and may fluctuate within the limits of 1.0 and 1.6 m/sec for sandy-loamy soils, 1.6 and 1.8 m/sec for gravel with a small quantity of boulders, and 2.0 and 2.5 m/sec for boulders.

Velocity computed by empirical formulas from the discharge appropriate to each opening is not valid for further computations and, therefore, it should be converted proportionally for the velocity admissible for the ground existing in the main channel and in the opening located in the valley flat.

Suppose that the mean velocity of 1.92 m/sec under bridge I has been obtained by empirical formulas after dividing discharges and 0.7 m/sec under bridge II. It was seen, on the other hand, from soil studies in the field, that a velocity of 1.8 m/sec can be tolerated under bridge I and only 1.3 m/sec under bridge II.

In this case, the velocity on the future bridge building site, as determined from the formula and amounting to barely 0.7 m/sec can be increased to the magnitude $1.92\,\frac{1.3}{1.8} = 1.38$ m/sec.

Then, the discharge section areas under bridges I and II can be computed from the formula:

$$P_1 = \frac{Q_1}{\mu\, 1.92} \qquad P_2 = \frac{Q_2}{\mu\, 1.38}$$

(5) After the discharge section area under bridge II has been determined, the opening is established.

(6) The size of an additional opening II is usually computed irrespective of the scour. When piers are very deeply founded, the coefficient of scour can be adopted as a magnitude not greater than 1.2.

(7) Artificial deepening of the channel is not to be recommended. It may in exceptional cases be envisaged, but not to a greater extent than 10 percent.

(8) If it appears that the opening thus computed varies from that initially assumed when determining the division of discharge. The computation should be repeated until a difference not greater than 5 percent is arrived at.

(9) If computations show that less than 20 percent of the entire discharge is passed through the additional opening, the idea of the additional opening should be abandoned.

(10) When designing several bridges in a valley flat, discharges are computed by a similar method, using a coefficient of irregularity between 1.15 and 1.20.

Thus the total discharge section area under bridges I and II, as computed, would be larger than under a single bridge in a main channel if no bridge had been built in the valley flat, because when the discharge section areas under bridge II were established a lower velocity had been adopted than under bridge I.

A large opening in a valley flat is obtained in this way and that is favorable for the discharge of water, since large openings or no openings at all should be applied in valley flats.

Designing Bridges Located in a Back Flow

If a bridge is designed near the mouth of a river, it can be placed within the limits of a back flow of a main river. That is, a high water can fill the channel and valley flat of a tributary reaching beyond the bridge built on such tributary.

The working conditions of the bridge are not usually deteriorated by the back flow. Such deterioration can happen only when rises occur simultaneously on the main river and on a tributary, and the fall of water on the main river is very rapid.

An elevation some dozen kilometers long can be caused on a tributary by a rise on a main river. The length of this elevation depends on the slope of the water level on the tributary, the difference between the ordinate of high water on the main river and on the tributary, etc.

In these cases, the ordinate of the high water should usually be additionally determined on the tributary within the limits of the bridge passageway but without the elevation caused by back flow. This is effected by means of a controlling cross section selected on the tributary, outside the limits of a back flow. The distance of the surface of the elevated water from the normal surface is determined in a controlling cross section, and the magnitude thus obtained is added to the normal level of water within the limits of the bridge passageway. The ordinate obtained determines the level of the high water in the bridge cross section of the river.

The discharge section area under the bridge on a controlling cross section is subsequently computed, and the discharge section area thus obtained is chosen in the cross section under the intended bridge. The size of the opening is first computed taking into consideration only the inflowing high water and disregarding the elevation caused by the back flow.

In the computation of a bridge located in the back flow it is necessary:

(a) to ascertain whether the opening would pass through the discharge existing in the unelevated sector,

(b) to determine the additional discharge caused by the back flow of a river, and to compute whether the discharge could be held by the opening during the water fall, the entire discharge section area under the bridge — i. e., together with elevation caused by the back flow — should be taken into account.

The discharge Q in cu m/sec during the fall of water may be computed approximately by the following formula:

$$Q = \frac{Fh\,1.5}{86\,400}$$

where:

F — water surface area (area of water within the limits of elevation above the bridge), in sq m;

h — daily fall of water in a main river, in m;

1.5 — coefficient of irregularity of fall of water;

86,400 — number of seconds in a 24-hour period.

6. Order of Computing a Bridge Opening

To compute the opening, a cross section of a river along the axis of an intended bridge should be drawn t o a scale depending on the size of the opening (Fig. 122).

The scale of length is usually adopted at between 1 : 500 and 1 : 5000, and the scale of height — between 1 : 50 and 1 : 200.

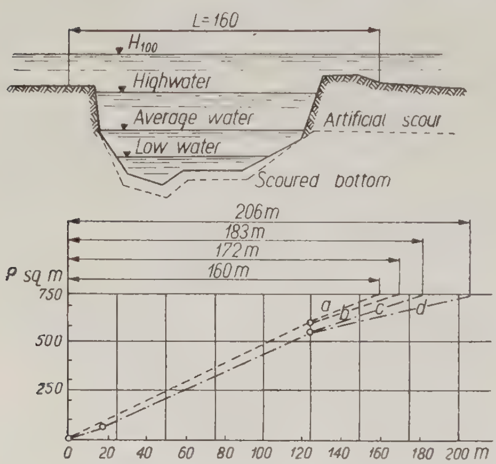


Fig. 122. Diagram for the computation of a bridge opening:

a-a — curve for cross section with scour and artificial scour, *a-b* — curve for cross section with scour and without artificial scour, *c-c* — curve for cross section with artificial scour and without natural scour, *c-d* — curve for cross section without scour or artificial scour

The ordinates of terrain and type of soil of the bottom in the main channel and valley flats are marked on the cross section.

The following elements should also be plotted on the cross section:

(a) water level adopted for computing the opening — e.g. with a probability of occurrence once in a 100 years ($H_{100} = H_{1\%}$);

(b) highest level of the spring ice drift (H_1);

(c) level of the mean summer water (H_o);

(d) level of the navigable or rafting water (navigable an drafting rivers).

Several variants of computations should be made at various depths of an artificial deepening and various coefficients of scour; the lines of mean scour for all these variants should be marked on the cross section.

In addition to the diagram of the cross section, it is advisable to draw a diagram of the correlation of the bridge opening with the discharge section areas for all variants of scour and artificial deepening. This diagram, introduced into the computations of bridges by Begam, facilitates the selection of the most favorable variant.

For this purpose, the stream cross section is divided into several parts, and individual areas are computed as between the bottom and the water surface adopted for computing the opening.

A diagram is then prepared of the correlation of an opening with the discharge section areas. The starting point of a diagram is usually selected on the border line of the designed bridge. Individual areas of the cross section formerly computed are summed up, and the values thus obtained are plotted on the diagram, the ordinates of which denote the discharge section areas and abscissae — the size of the opening. Points thus established are connected by an undulating line (Fig. 122).

The scale of a diagram is selected so that this line is inclined at an angle of about 45° to the horizontal.

The line is traced up to the point whose ordinate equals the necessary discharge section areas under the bridge. The clear span of the opening L is determined by the corresponding abscissa.

The entire main channel of the river should be included within the sector for which transverse areas are computed. Otherwise, the position of the starting point should be changed.

After drawing the lines of correlation of the bridge opening with the discharge section area for individual variants, a variant is selected to give the smallest opening L.

The variant adopted should meet all the technical requirements at minimum cost of building the bridge passageway.

The size of the bridge opening is markedly influenced by the shape of that cross section of the stream in which the bridge is located. The opening will be smallest for that identical area of the cross section under the bridge for which the mean depth of the stream under the bridge is highest.

Approximate Method of Computing the Bridge Opening

An approximate size of the bridge opening L can be computed from Sribny's formulas.

Rivers as concerning which deep valley flats play a considerable part in passing the discharge of water

The size of the bridge opening for such rivers can be computed from the following formulas:

(a) irrespective of scour:

$$L = 0.95\, l_1 + 0.04\,(l_2 + l_3)$$

(b) taking scour into account (coefficient $K = 1.2$):

$$L = 0.80\, l_1 + 0.033\,(l_2 + l_3)$$

The size of opening for rivers of this type can be computed on the basis of the following formulas:

(a) irrespective of scour:

$$L = l_1 + 0.08\,(l_2 + l_3)$$

(b) with scour taken into account (coefficient $K = 1.2$):

$$L = 0.85\,l_1 + 0.066\,(l_2 + l_3)$$

where l_1, l_2, l_3 = respective widths of the main channel and the left and right hand valley flat.

Formulas which disregard the scour should be used when designing wooden bridges. The results obtained should be multiplied by the coefficient 1.2, in view of the fact that a general increase in the width of piers causes contraction of single water streams.

Examples of Computing Bridge Openings

Example 1

A bridge opening is to be computed on a navigable river for a discharge $Q_{100} = 6{,}300$ cu m/sec. The intended bridge is to have 7 spans supported by 6 stone piers having semicircular front parts. Navigation requirements necessitate that the bridge shall have openings 80 m in diameter. Bottom ground — fine sand.

The cross section of the river along the axis of the intended bridge is shown in Figure 123.

Ordinate of the level of water with a probability of occurrence once in a 100 years — $H_{100} = 111.73$ m. Discharge with the same probability once in a 100 years in the main channel and in valley flats — $Q_{100} = 6{,}300$ cu m/sec.

The discharge section area in the main channel and the valley flats before the building of bridge $P_w = 7{,}621$ sq m.

Discharge of the main channel $Q'_{100} = 5{,}700$ cu m/sec.

Discharge section area in the main channel $P'_w = 4{,}882$ sq m.

Width of the main channel $B = 680$ m.

Mean depth in the main channel:

$$h_o = P'_w : B = 4882 : 680 = 7.2\text{ m}$$

Mean velocity of water in the main channel and valley flats prior to the building of the bridge:

$$v_1 = Q_{100} : P_w = 6300 : 7621 = 0.83\text{ m/sec}$$

Mean velocity of water in main channel without structures:

$$v_2 = Q'_{100} : P'_w = 5700 : 4882 = 1 \cdot 17\text{ m/sec}$$

Admissible velocity v_d of water, depending on the type of soil and the depth h_0 taken from the Lishtvan Table (Table 82) amounts to:

$$v_d = 1.16 \text{ m/sec}$$

Coefficient of contraction μ taken from Table 80:

$$\mu = 0.98$$

Required discharge section area under the bridge:

$$P = \frac{Q_{100}}{v_d \, \mu} = \frac{6300}{1.16 \times 0.98} = 5540 \text{ sq m}$$

Area occupied by 6 bridge piers:

$$P_p = 190 \text{ sq m}$$

Area existing under the bridge without the area occupied by bridge piers:

$$P' = P'_w - P_p = 4882 - 190 = 4692 \text{ sq m}$$

The area existing under the bridge P' is smaller than the required discharge section area P. The difference is made up by admitting the bottom scour.

Coefficient of scour:

$$K = P : P' = 5540 : 4692 = 1.18$$

The point of the cross section situated at 0 + 0.387 km is adopted as the beginning of the bridge.

Summing up the individual areas of the cross section situated to the right from the point adopted as the beginning of the bridge, it is found that the required discharge section area under the bridge — 5540 sq m — will be included, after scour, between 0 + 0.387 km and 1 + 0.007 km — i.e., the bridge opening between the front walls of the abutments amounts to:

$$L = 1007 - 387 = 620 \text{ m}$$

The opening L is computed including the widths of piers.

The velocity existing under the bridge prior to the scour of the channel:

$$v_2 = \frac{Q_{100}}{\mu \, P'} = \frac{6300}{0.98 \times 4692} = 1.37 \text{ m/sec}$$

Elevation above the bridge amounts to:

$$h = \lambda \, (v_2^2 - v_1^2) = 0.1 \, (1.37^2 - 0.83^2) = 0.12 \text{ m}$$

Example 2

Compute the approximate size of a bridge opening for the cross section given in Example 1 (Fig. 123).

The Sribny formula for rivers with deep valley flats participating to a con-

siderable extent in passing water discharge is used here, taking into account coefficient of scour 1.2.

Since the bridge cross section of the river has a valley flat on one side only, the Sribny formula has here the following form:

$$L' = 0.80\, l_1 + 0.033\, l_2$$

where:

l_1 — width of main channel = 680 m

l_2 — width of valley flat = 1,060 m.

The following width of the opening is obtained after substituting the above values in the formula:

$$L = 0.8 \times 680 + 0.033 \times 1060 = 544 + 35 = 579 \text{ m}$$

Since piers occupy $6 \times 4.0 = 24$ m, the approximate length of the bridge opening between the abutments will amount to:

$$L = 579 + 24 = 603 \text{ m}$$

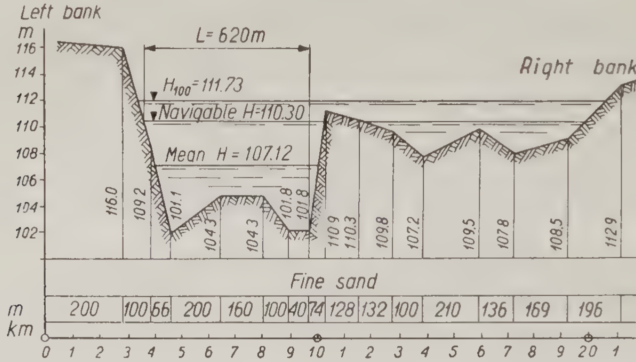


Fig. 123. Cross section of a river along the axis of an intended bridge

When compared with the size of opening $L = 620$ m computed by the more accurate method, the error amounts to about 3 percent.

CHAPTER IX

COMPUTING OPENINGS OF SMALL BRIDGES AND CULVERTS

1. Water Discharge Passing Through Small Openings

An uncontracted stream has, prior to the building of a bridge, fixed velocities, widths and depths depending on the discharge and slope of the channel.

For passing the highest discharge an opening slightly smaller than the width of the water surface in a free channel is usually adopted. A contraction of discharge section area then appears, causing elevation of water above the structure and an increase in the velocity of the water streams flowing through the opening.

The depth of the elevation can be lower than, equal to or higher than the height of an opening or — in other words — an opening can be free, partially flooded, or submerged.

When an opening is submerged, the discharge of water takes place inside it under pressure. A greater quantity of water flows through such an opening than through a pressureless one, identical in size and under identical conditions.

Shapes of Openings

Openings of various shapes have hitherto been applied in bridge building. At present, the most often built culverts have rectangular openings and those intended for low discharges — circular openings.

Rectangular openings, as compared with the previously used oval openings, are cheaper and have a greater capacity. As regards stability, particularly favorable, because involving a diminished thickness of walls, are rectangular stone culverts with a reinforced concrete plate so constructed as to be able to resist the pressure of earth exerted on culvert walls. The walls of a culvert can be strutted off by means of:

(1) the plate ends being cut aslant (Fig. 124a);

(2) resting the plate against the protruding part of the culvert wall (Fig. 124b);

(3) resting the thickened plate ends against the culvert wall (Fig. 124c, d);

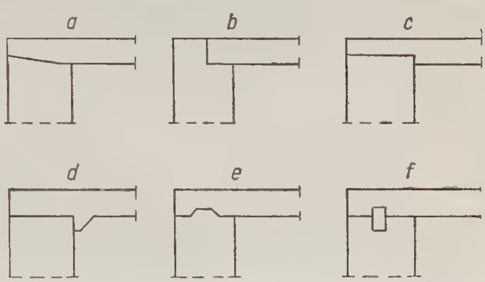


Fig. 124. Details of joining of plates with walls of a culvert

(4) making stubs in the upper part of a culvert wall and cutting corresponding notches in the plate (Fig. 124e);

(5) inserting sectors of pipes 20 cm long and 10 cm in diameter in the plate and in the culvert wall (Fig. 124f).

Circular openings should be applied for culverts with insignificant discharge. Structures with circular cross section are usually built of reinforced concrete. They may consist of separate elements, mass produced in factories or on building sites. A high quality of production is thus achieved at low cost and in a short time.

Structures with circular cross sections are particularly advantageous from the hydraulic point of view if the water discharge takes place under pressure.

It will be seen that structures with rectangular and circular openings have many advantages and should, therefore, be used in practice.

Culverts with several openings

To pass water when the height of an embankment is too low, or for other reasons, culverts with several openings differing or identical in size are sometimes used. Building culverts with several openings should, however, be avoided since they are disadvantageous from the hydraulic point of view.

The division of the entire discharge for openings of various sizes — and particularly small ones — is not easy to determine and, therefore, some of them may be overloaded with an excessive discharge adversely influencing the stability of a structure.

It may sometimes be observed in practice that in culverts with several openings, bottom scour appears below some of such openings. In addition it has been ascertained by way of observation that at a certain moment, even in a culvert having several openings of identical size, a greater quantity of water starts to flow through some of them than through others, although the general discharge is constant. This causes an increase in the velocity of water in overloaded openings and in the channel scour below them, which in turn may contribute to scour of the entire structure.

For such reasons, building culverts with several openings should be avoided and, if possible a simple low, wide opening should rather be selected.

Pressureless openings

The elevation of water above the structure causes an increase in the velocity within the opening and, therefore, structures with flooded openings are not always used. Great velocities arise in such openings, which may cause bottom scour or destruction of the material of which the structure is built.

The use of pressureless openings ensures a higher stability in the structure, although the capacity of such openings is lower.

If a culvert is long and has a small opening, it may be assumed that water discharge in pressureless openings takes place similarly to broad crested weirs.

Pressure openings

When designing culverts, it is very often assumed that they are pressureless since their openings will not be submerged. However, as can be observed in practice, some culverts — computed according to this assumption — do work sometimes with their openings flooded.

In a catchment basin, there may occur a rise higher than that assumed for determining a culvert opening. To compute the elevation of embankment crests, discharges are adopted with a probability of occurrence lower than that applied when determining the series of openings in structures located in those embankments.

Consequently the flooding of culvert openings is always liable to happen. In principle the openings of outlets in hydraulic structures for elevating water level also work with submerged inlets. Culverts should, therefore, be computed on the assumption that their inlet will be submerged and for this reason they should be properly built.

It was assumed until recently, that an opening with a submerged inlet works with its full cross section regardless of the shape of the inlet. This is an incorrect assumption, because the discharge of water under pressure in openings with unstreamlined inlets does not take place with the full cross section but in a manner depicted in Fig. 125a. To ensure that the full cross section of an opening works, the inlet should be streamlined in shape. (Fig. 125b).

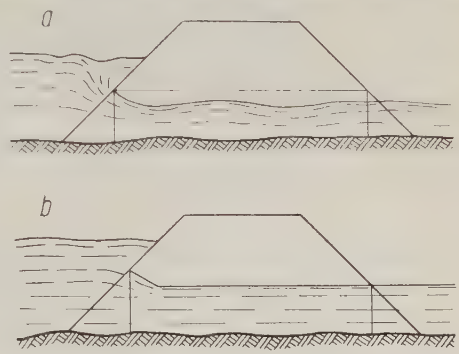


Fig. 125. Water discharge for pressure culverts

a – culvert working with a part of its cross section, *b* – culvert working with full cross section

The discharge through a pressure opening was studied by the author of the present work at the Hydro-Laboratory of the PIHM (National Hydro-Meteorological Institute). The following conclusions can be drawn on the basis of these studies:

(1) Given an inclined embankment slope or a vertical wall above the inlet of a submerged opening, the opening works with its full cross section if the intake is only slightly flooded;

(2) as the discharge volume is increased, there comes a moment when streams of water begin to detach from the upper surface of the opening conduit — at first near the inlet and then, gradually, over the whole length up to the outlet;

(3) the height of an elevation at which the opening under apparently identical conditions begins to work with only a part of its cross section, is not constant; sometimes, also, the detachment of streams may not appear at all, irrespective of an increase in the height of the elevation;

(4) the detachment of streams from the upper surface of the inlet is caused by the air sucked into the opening on the side of the inlet, with simultaneous formation of characteristic eddies on the surface of the elevated water;

(5) water working with only a part of a cross section reaches between 0.6 and 0.8 of the height of the opening and there appears an increase in elevation as compared with that when the opening is working with its full cross section;

(6) an opening where there is increased height at the intake following the upturning of its upper surface at an angle greater than 45° shows a tendency to work with a part of its cross section;

(7) when an angle approaching 45° is used, the detachment of streams from the upper surface of the inlet rarely appears, although such appearance is possible;

(8) when the upper surface of the inlet is upturned at an angle of about 30° to the axis of a culvert, no detachment of streams could be observed on a model irrespective of the magnitude of elevation.

The above conclusions show, that upturning the upper surface of the inlet of a rectangular opening at an angle approximately 30° produces work with a full cross section, regardless of the magnitude of elevation.

The limits, within which this angle can be changed should be determined by subsequent detailed research, which should also establish the influence of the magnitude of elevation on the magnitude of the inclination angle of the upper surface of the opening.

2. Influence of the Shape of Inlet and Outlet on the Water Discharge

The magnitude of discharge passing through openings with identical discharge section area and constant flow is not identical, but depends on the shape of the opening, which causes larger or smaller losses.

Among the principal types of inlets so far used are:

(a) inlet without wings called a flanged inlet;

(b) inlet with slanting (obtuse) wings;

(c) inlet with perpendicular wings;

(d) inlet with parallel wings.

Culverts with a rectangular cross section

On examining the inlets of culverts with a rectangular cross section, it was found that the most unfavorable from the hydraulic point of view was the flanged inlet (Fig. 126a) which caused the greatest contraction and the highest losses.

The most suitable, on the other hand, was an inlet with slanting wings placed at an angle of 30° (Fig. 126b), which caused a 10 to 20 percent increase in the discharge over and above the openings with differently shaped wings.

The application of an inlet with wings slanting at less than 30° causes greater losses. Wings slanting at a greater angle (up to 45°) give no visible hydraulic advantages, although the building of such wings is more expensive.

It is also important to design an outlet of such a shape as to cause the spread of water after it leaves the opening without scouring the channel banks, and forming eddies and turbulences.

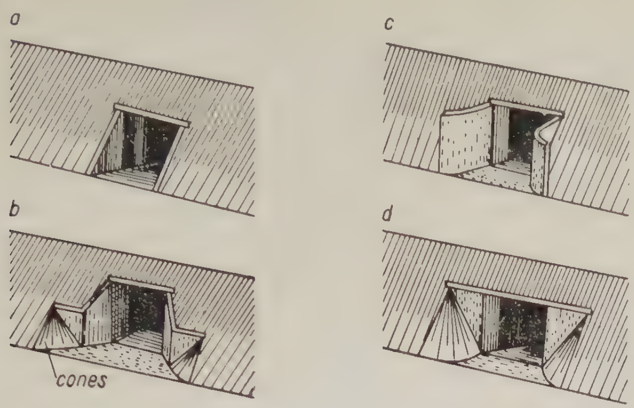


Fig. 126. Types of culvert inlets with rectangular cross section: *a* — oblique inlet, *b* — open, *c* — perpendicular, *d* — parallel

In practice the same types of wings have been applied at the outlets and the inlets, and it was expected that it would suffice to slant them against the axis of an opening at an angle of about 10°. It was found by way of laboratory studies that both hydraulically and economically the most advantageous is an outlet with wings having no cones and slanting at an angle of 30° against the axis of the opening, because in this case a decrease in the velocity of water streams below the outlet is obtained.

The following phenomena were observed during studies of discharges passing through culverts with rectangular cross sections:

(1) undisturbed and uniform distribution of the streams in the free weir takes place with wings slanting at angles between 21° and 45°, while at 20° streams spread to the embankment, thus proving that this angle is too small;

(2) the distribution of water of streams with a submerged weir is smaller and, at the same time, gives rise to contrary currents which not only approach an embankment but partly get into the inlet;

(3) with a free outlet the streams spread symmetrically and are distributed fanwise; after an elevation of tailwater to half of the height of an opening, the streams of water leaving the outlet spread only insignificantly and, subsequently, after flowing a short distance, they turn sharply to the side, and the whole stream assumes an asymmetrical position.

(4) on leaving the openings, streams obtain their maximum velocity after flowing a distance approximately equal to the double clear span of

structures and exceed the theoretically computed velocity inside openings; these data are presented in Table 84.

It should be remembered that in the case of a rectangular submerged inlet, in addition to the slanting wings, the upturning of the upper edge of the inlet

Table 84

Excess of the Theoretically Computed Velocity of Streams

Type of weir	Outlet wing slant	Excess of theoretically computed velocity
free	0–20°	1.3 times
„	30–45°	1.2 times
Submerged	0–20°	1.2 times
„	30–45°	1.1 times

referred to above should be adopted in order to facilitate the work of an opening with its full cross section.

Culverts with a Circular Cross Section

The discharge capacity of a pressure circular opening may be increased by designing inlets having specially streamlined shapes.

In 1948 and 1949, studies towards this end were undertaken at the Soviet Scientific Road Research Institute (DORNII). Types of inlets shown in Figure 127 were the subject of investigations to establish which of these types involves the lowest resistances and is at the same time simple in construction and cheap.

The types shown in Figures 127a and 127b were tested on the appearance of tailwater and a slope equal to 0; it was found, that the occurrence of a depth of tailwater to 0.4 d (d — diameter of a pipe) and the reduction of the slope to 0 exert a quite insignificant influence on the capacity of pressure penstocks.

The correlation discharge curves with height of elevation are shown in Figure 128. These curves were obtained after invesitgating pressure pipes 15 cm in diameter and 200 cm long. Table 85 shows the assembled discharges passing through pipes at various elevations for which the capacity of pipes with flanged inlets working under pressure with unfilled cross sections is assumed as 100 percent.

This table proves that at the most commonly adopted elevation equal to $2d$, the difference in capacities as between extended and conical inlets is insignificant.

With a view to selecting the most appropriate type of inlet, comparisons were also made of the costs and conditions of producing pipes with a diameter $d = 1.0$ m and length 2.7 m, with various types of inlets. For such assumptions it was computed that the volume of material required for manufacturing a pipe of reinforced concrete with a streamlined extended inlet amounts to 0.89 cu m; with an elliptical inlet — 1.40 cu m; and with an extended circular inlet — 1.13 cu m.

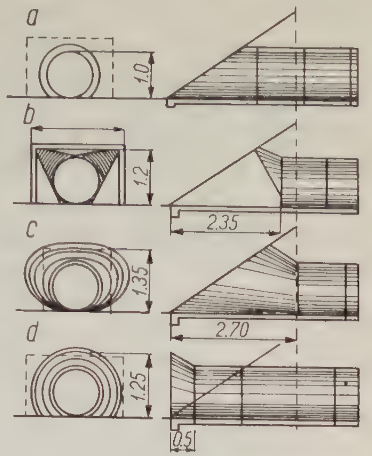


Fig. 127. Types of culvert inlets with circular cross section: *a* — flange inlet, *b* — open, *c* — elliptical, *d* — circular extended

Since some types of inlets are more complicated, pipes should also be compared as regards facility of construction.

The streamlined extended inlet is so constructed that the flat walls of slanting wings are sloped to achieve better counteraction against earth pressure. Both wings are connected with the flat surface in the upper part of an inlet by means of small conical surfaces. This inlet has the largest number of flat surfaces, and due to this advantageous form the shuttering of the structure presents no great difficulties.

Shuttering a conical inlet is exceptionally complicated, because it must have variable curvatures over a circumference of great length.

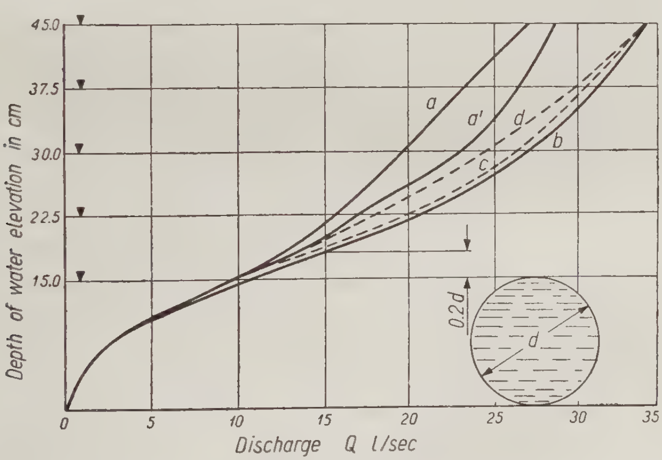


Fig. 128. Correlation curves for discharge and magnitude of waterhead elevation flange: *a* — in culverts with the flange inlet working with a part of the cross section, *a'* — with the flange inlet and opening working with full cross section, *b* — with open inlet, *c* — with elliptical inlet, *d* — with extended flange inlet

Discharge in Pipes with Submerged Inlets

Height of elevation	Flanged inlet working: with unfilled cross section *a*	Flanged inlet working: with filled cross section* *a′*	Streamlined inlets: Slanting *b*	Streamlined inlets: Elliptical *c*	Streamlined inlets: Circular extended *d*	Remarks
1.0 d	100	100	111	109	103	* Work with an artificially induced full cross section
1.5 d	100	111	134	128	117	
2.0 d	100	116	136	133	124	
2.5 d	100	114	133	130	128	
3.0 d	100	107	127	127	125	

Shuttering a circular extended inlet is less complicated; however, the fact that the intersections of circles with the surface of the bottom form parabolas makes shuttering somewhat difficult.

Comparing the above variants of inlets from every point of view previously discussed, it seems necessary to disqualify conical and extended circular inlets. The extended streamlined inlet proves to be the most advantageous and should, therefore, be applied in practice.

For small discharges and depths, a crease inlet may also be used as being the simplest and cheapest to make.

3. Computing the Openings of Small Bridges

The clear span of openings depends on the discharge, on the velocity admissible in an opening which is conditioned by the type of soil, and the size of the discharge section area within the limits of a structure.

The size of an opening diminishes as the admissible velocity inside a structure increases.

The theory of computing the bridge and culvert openings is primarily based on the Bernoulli equation and on investigations conducted by other known hydraulic experts.

The following order should be observed in computing openings of small bridges:

(a) determine maximum depth of a stream in a free, structureless channel for the assumed discharge;

(b) fix the magnitude of the admissible velocity v under the bridge in correlation with the type of ground or artificial reinforcement of a channel;

(c) determine whether the opening is to work according to the principle of a submerged or free weir;
(d) compute the required clear span of the opening.

Determining Maximum Stream Depth

To determine maximum depth a in a free, structureless channel of a stream, the water level in a cross section of the channel above the intended structure should be established by trials.

For this purpose, any level of water surface in a cross section may be assumed, and the velocity v found from the Manning formula:

$$v = \frac{1}{n} R^{2/3} i^{1/2}$$

where:

R — numerical value of the hydraulic radius, in m;
i — slope of the stream bed, expressed as a decimal fraction;
$\frac{1}{n}$ — coefficient of roughness to the Manning formula (Table 16).

Subsequently, we determine the discharge Q in the cross section F for the height of water level adopted and the computed velocity v:

$$Q = F v$$

If the discharge thus determined does not differ from the existing one by more than 5 percent, the water level adopted may be considered as valid, and the maximum depth a is determined from the cross section of a channel.

In a cross section with a distinctly outlined main channel and side valley flats, or with a large difference in the depths of individual parts of a cross section, the discharge is computed for each part separately, and the partial volumes thus obtained are summed up.

For flat beds, whose width is at least 30 times greater than the depth, we can replace in the Manning formula the hydraulic radius R by the mean depth t equal to the quotient of the area of the cross section F and the width of a water surface B in such cross section.

Determining Admissible Velocities Under a Small Bridge

A mean admissible velocity of water in the opening should be fixed after determining the depth of water in a free channel. This velocity depends on whether the bottom of a structure is to be left in a natural state or some kind of revetment applied.

The use of a channel revetment, even allowing for possible repairs, is usually less expensive than increasing the span of an opening. Therefore, allowance is

often made for a larger velocity in an opening and, consequently, a gain is achieved by reducing the size of an opening.

A channel should be revetted under the bridge along its entire width and length, which is equal to the distance between the lines of intersection of the road declivities with the terrain on both sides of a bridge.

Admissible velocities depending on the type of soil under a bridge can be adopted from Tables 77 and 78.

Velocities admissible for reinforced channels are presented in Table 79.

Determining the Size of the Opening of a Small Bridge

The size of openings of small bridges depends to a considerable extent on the velocity which can be admitted under a bridge and, therefore, it is our aim that this velocity shall be higher than in a free, structureless channel.

If, due to local conditions, a higher velocity than that existing in a free channel cannot be admitted under a bridge, and a bed revetment is not envisaged, the opening is designed with its width equal to the width of water surface in the cross section of the stream with the discharge adopted in computations.

The contraction of a stream and a design of bridge opening smaller than the width of the water surface is possible if the admissible stream velocity under the bridge is greater than that existing in a free channel.

The discharge of water passing through the openings of small bridges after the contraction of the stream is similar to the discharge spilling over the broad crown, which can take place according to the principle of either a free or submerged opening.

In a free opening, the water level below the structure does not exert any influence on the water level in an opening, and can be lower than, equal to or slightly higher than the level.

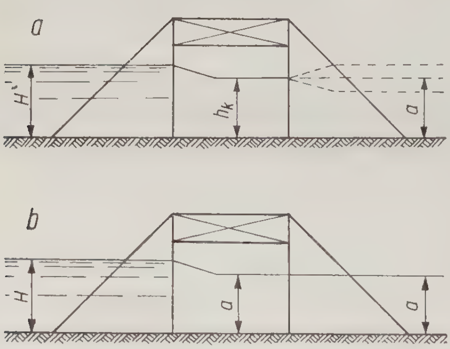


Fig. 129. Water discharge under a bridge: *a* — free weir, *b* — submerged weir

In this case, a critical depth h_k becomes stabilized in the opening and is equal to:

$$h_k = \sqrt[3]{\frac{Q^3}{b^2 g}}$$

where:

h_k — critical depth, in m — i.e. a depth at which the unit power of the flowing water is the smallest;

Q — discharge, in cu m/sec.;

b — size of opening, in m;

g — acceleration due to gravity = 9.81 m/sq sec.

A free opening arises when the depth of water without a headwater $a \leqslant 1.3\, h_k$ (Fig. 129 a).

In a free opening, the level of water in an opening depends on the height of the water level in an outleading channel. The submerged opening appears at the depth of water below a structure a greater than 1.3 h, and the difference between the depth in an opening and below it may be disregarded in computations.

Computing an Opening of a Small Bridge with a Free Opening

The following correlation appears with a free opening:

$$a \leqslant 1.3\, h_k$$

After transforming the equation of the critical velocity $v_k = \sqrt{g h_k}$, the magnitude of the critical depth is obtained:

$$h_k = \frac{v_k^2}{g}$$

And after substituting this value in the general formula for computing the discharge $Q = Fv$, we obtain:

$$Q = Fv_k = b_k h_k v_k = b_k \frac{v_k^2}{g} v_k = \frac{b_k v_k^3}{g}$$

therefore, the width of a stream b_k with critical depth h_k amounts to:

$$b_k = \frac{Qg}{v_k^3}$$

Due to the contraction caused by the bridge pillars, the formula for computing the width of an opening of any shape in a free opening has the following form:

$$b = \frac{Qg}{\mu v_k^3} = \frac{Qg}{\mu v^3}$$

where:

b — width of an opening, in m;

Q — adopted discharge in cu m/sec,

g — acceleration due to gravity = 9.81 m/sq sec.;

v_k — critical velocity in m/sec, adopted as being equal to the admissible velocity v for a revetted or unrevetted stream channel under a bridge (Tables 78, 79, 82);

u — coefficient of contraction depending on the shape of abutments (Table 86).

The following formula is used for computing the width of an opening if a bridge consists of several bridge spans:

$$b = \frac{Qg}{\mu\, v^3} + Nd$$

where:

N — number of indirect bridge piers;

d — width of an indirect pillar;

v — velocity admissible under the bridge, in m/sec,; the remaining denotations are the same as in the previous formula.

Table 86

Magnitude of the Coefficient of Contraction μ

No.	Shape of Abutments	Size of openings in m			
		4.0	6.0	10.0	> 10.0
1	With longitudinal stone dikes of a height flush to the surface of water	0.95	0.95	0.96	0.97
2	With cones	0.90	0.90	0.90	0.95
3	With slanting wings	0.85	0.85	0.85	0.90
4	Protruding from cones	0.80	0.80	0.80	0.85

The critical depth h_k of a stream under the bridge is computed for the channels with rectangular or wide trapezoidal cross section from the formula:

$$h_k = \sqrt[3]{\frac{Q^2}{g\, b_k^2}} = \frac{v_k^2}{g} = 0.1\, v_k^2$$

The following formula is used for computing critical depth h_k in the narrow deep trapezoidal channels:

$$h_k = \frac{b_k - \sqrt{b_k^2 - 0.4\, s b_k v_k^2}}{2s}$$

where:

b_k — the width of a stream in a critical state, in m;

s — inclination of the slopes of the cross section of the stream under the bridge ($1 : s$); if, for instance, the inclination of slopes amounts to $1 : 2$, $s = 2$; the remaining denotations are the same as in the previous formulas.

The width of the stream bottom for the trapezoidal channels is equal to:

$$B = b - 2\, s h_k$$

The depth of the stream before bridge H in m may be calculated from the formula

$$H = h_k + \frac{v_k^2}{2g\,\varphi^2} - \frac{v_c^2}{2g}$$

where:

φ — coefficient of velocity, taking into account the loss of water power near the opening inlet (Table 87);

v_c — velocity of a stream in m/sec above the structure in the cross section where depth equals H;

at $v_c \leqslant 1$ m/sec., the last expression — i. e. $\frac{v_c^2}{2g}$ — can be disregarded, because its value amounts then to $\leqslant 0.05$ m; at $v_c > 1$ m/sec, the equation determining H is solved by substitution, because v_c depends on H.

Table 87

Magnitudes of the Coefficient of Velocity φ

No.	Shape of Abutments	φ
1	With longitudinal stone dikes of a height flush to the surface of water	0.95
2	With cones	0.90
3	With slanting wings	0.90
4	Protruding from cones	0.85

Example

Compute the opening of a wooden bridge on pile bents made of piles 25 cm in diameter and with cones inclined as $1 : s = 1 : 1.5$. The discharge Q amounts to 6 cu m/sec, and the velocity admissible in the revetted channel under the bridge $v_k = 2.5$ m/sec.

The coefficient of contraction $\mu = 0.9$ for abutments with cones under the bridge is adopted from Table 86, and the coefficient of velocity $\varphi = 0.9$ — from Table 87.

The width of free water surface amounts to:

$$b_k = \frac{gQ}{v_k^3} = \frac{9.81 \times 6}{2.5^3} = 3.8 \text{ m}$$

The width of an opening after designing one pile bent under the bridge and applying the coefficient of contraction will be equal to:

$$b = \frac{b_k}{\mu} + Nd = \frac{3.8}{0.9} + 0.25 = 4.47 \text{ m}$$

The critical depth h_k amounts to:

$$h_k =$$

$$= \frac{b_k - \sqrt{b_k^2 - 0.4\, sb_k\, v_k^2}}{2s} = \frac{3.8 - \sqrt{3.8^2 - 0.4 \times 1.5 \times 3.8 \times 2.5^2}}{2 \times 1.5} = \frac{3.8 - 0.44}{3} =$$

$$= 1.12 \text{ m}$$

The width of the bottom of the stream cross section under the bridge is as follows:

$$B = b - 2sh_k = 4.47 - 2 \times 1.5 \times 1.12 = 1.11 \text{ m}$$

A free weir will appear under the bridge at the depth:

$$h_n = 1.3\, h_k = 1.3 \times 1.12 = 1.46 \text{ m}$$

In the event of greater depths under the bridge, a submerged weir will arise.

The depth of the elevated water before the structure at $v_c < 1.0$ m/sec amounts to:

$$H = h_k + \frac{0.1\, v^2}{2\, \varphi^2} = 1.12 + \frac{0.1 \times 2.5^2}{2 \times 0.9^2} = 1.51 \text{ m}$$

Computing an Opening of a Small Bridge with Suberged Weir

The following correlation occurs at a submerged weir:

$$a > 1.3\, h_k$$

In such cases, the depth of the stream under a bridge is adopted as being equal to the depth of stream a in the outleading channel (Fig. 129 b).

For a submerged weir, the width of the rectangular opening is computed from the formula:

$$b = \frac{Q}{\mu\, av} + Nd$$

where:

a — depth in the outleading channel in m; the remaining denotations are the same as in the preceding formulas.

If the cross section of a channel under the bridge is trapezoidal in shape, the width b is a median line of a trapezoidal active cross section under the bridge at half the depth a.

The depth of a stream above the bridge is computed from the formula:

$$H = a + \frac{v^2}{2g\, \varphi^2} - \frac{v_c^2}{2g}$$

If the velocity of the inflowing water $v_c \leqslant 1.0$ m/sec., the last expression of the above equation can be disregarded. For $v_c > 1.0$ m/sec, the depth above the bridge H is computed by substitution.

4. Computing Openings of Culverts

In concrete or stone culverts, very high velocities can be allowed, reaching 15.0 to 20.0 m/sec. Such velocities, however, would cause too great an elevation and involve the necessity of building strong revetments of a bed over great lengths below a culvert. Therefore, in designing culverts an admissible velocity of about 5 to 6 m/sec is adopted, because it is easier to pass from such a velocity to the velocity in a stream channel of 0.5 to 1.5 m/sec, which may be admitted in natural channels without fear of any considerable scour.

The adoption of a higher velocity in a culvert than in a natural channel facilitates designing a clear span of an opening smaller than the width of a stream. In such culverts, the discharge of water can be passed with either a free or a submerged inlet and the culvert then works under pressure.

Pressureless Culverts

If a culvert is a pressureless one, the stream flows over its entire length, as in channels with free water surface, and the discharge is passed according to the principle of a free weir with a wide crest (Fig. 130 a).

A culvert is a pressure one when water depth above the structure H is $\leqslant 1.2\ h$ at ordinary inlets, or $H \leqslant 1.4\ h$ at streamlined inlets where h is the height of a culvert.

The depth of water in a culvert near its inlet amounts to $h_c = 0.9\ h_k$, and above the inlet it depends on the slope and length of the culvert.

If the slope of the culvert bottom I is greater than the critical slope I_k, the depth of water above the inlet is equal to the depth of a structureless stream h_o, and in the event of a slope $I \leqslant I_k$, the critical depth h_k is fixed above the inlet.

Openings of pressureless culverts, consideration being given to their admissible filling k at the inlet, are computed from the formula:

$$k = \frac{H}{h}$$

Computing Culvert Openings with Rectangular Cross Sections

The clear span b in m of a culvert with a rectangular opening may be computed as for a small bridge from the formula:

$$b = \frac{Qg}{\mu v^3} \text{ or } b = \frac{Q}{1.7\,\mu H^{1.5}}$$

The following magnitudes of the coefficient μ can be adopted for the rectangular opening:

for a culvert with slanting wings $\mu = 0.90$
for a culvert with parallel wings $\mu = 0.85$
for a flanged culvert without wings $\mu = 0.80$

The depth of water above the structure H may be computed by the trial method using the above presented formulas for computing small bridges working according to the principle of a free weir with wide crown.

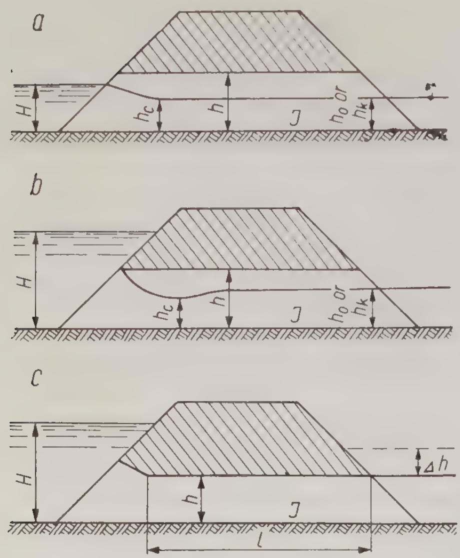


Fig. 130. Scheme of water discharge in a culvert: *a* — pressureless culvert, *b* — pressure culvert working with a part of the cross section, *c* — pressure culvert working with full cross section

The depth and velocity of water above the outlet in a culvert having a bottom slope greater than the critical slope are determined by choosing normal depth h_o of the uniform movement and computing the velocity from the formulas:

$$Q = Fc\sqrt{RI}$$

$$v = c\sqrt{RI}$$

In culverts designed for slopes equal to or smaller than the critical ones, the depth of water above an outlet is equal to the critical depth, which is computed from the formula:

$$h_k = \sqrt[3]{\frac{Q^2}{gb^2}}$$

Computing Culvert Openings with Circular Cross Section

The diameter d of a culvert with circular cross section is determined from the formula:

$$d = \left(\frac{Q}{1.4\,\mu}\right)^{2/5}$$

The following coefficients of contraction may be adopted for circular cross sections:

for a culvert with slanting wings $\mu = 0.85$

for a culvert with perpendicular wings $\mu = 0.80$
for a flanged culvert without wings $\mu = 0.75$

The depth h_o at the outlet and the critical depth h_k in the circular openings are computed by means of Table 88.

Table 88

Values for Computing Circular Openings

Degree of filling of an opening $h_o : d$	Critical function $\frac{Q^2}{gd^5}$	Ratio of moduli of discharge $K_o : K_d$	Ratio of moduli of velocity $W_o : W_d$
0	0	0	0
0.05	0	0.004	0.184
0.10	0	0.017	0.333
0.15	0	0.043	0.457
0.20	0.001	0.080	0.565
0.25	0.005	0.129	0.661
0.30	0.009	0.188	0.748
0.35	0.016	0.256	0.821
0.40	0.025	0.332	0.889
0.45	0.040	0.414	0.949
0.50	0.060	0.500	1.000
0.55	0.088	0.589	1.045
0.60	0.121	0.678	1.083
0.65	0.166	0.766	1.113
0.70	0.220	0.850	1.137
0.75	0.294	0.927	1.152
0.80	0.382	0.994	1.159
0.85	0.500	1.048	1.157
0.90	0.685	1.082	1.142
0.95	1.035	1.087	1.108
1.00		1.000	1.000

Table 88 presents:

(a) degree of the filling of an opening $= \frac{h_o}{d}$,

(b) value of critical function $= \frac{Q^2}{gd^5}$,

(c) ratio of modulus K_o of the existing discharge to the modulus K_d of the discharge which can be passed through the opening, where:

$$K_o = \frac{Q}{\sqrt{I}}, \quad \text{and} \quad K_d = 24\, d^{8/3}$$

(d) ratio of moduli of velocities W_o to W_d, where:

$$W_o = \frac{v}{\sqrt{I}}, \quad \text{and} \quad W_d = 30.5\, d^{2/3} \; (d = \text{culvert diameter in m}).$$

Pressure Culverts

In pressure culverts — i.e. with their opening fully submerged — the flowing water may fill the opening entirely or partially, depending on the shape of inlet applied.

The computation of pressure culverts with partially filled openings is somewhat different from the computation of culverts entirely filled.

Partial Filling of the Culvert Opening

The partial filling of the culvert opening (Fig. 130 b) appears with the adoption of inlets of unstreamlined shape and depth H of elevation above the culvert greater than 1.2 h.

Near the inlet of a culvert the depth h_c is fixed smaller than the critical depth h_k, namely:

$$h_c = \mu\, h < h_k$$

where:

h — height of culvert in its clear span,

μ — coefficient of contraction, whose value for rectangular openings amounts to $\mu = 0.60$ and for circular openings $\mu = 0.65$.

Pressure culverts working with a part of their cross section are computed for admissible water velocities by using formulas for the discharge passed through the opening in a wall.

Since water velocities occurring near the partially filled outlets of openings are considerable, the fluctuations in water level below the culvert do not exert any influence on the depth of water elevated above the culvert.

The depth of water in a culvert near its outlet depends on the slope of the culvert bottom. At botton slopes $I > I_k$, the normal depth h_o appears at the outlet and this corresponds with the slope of the culvert bottom, while at $I \leqslant I_k$, the depth at the outlet is equal to the critical depth h_k.

The computation of the velocity of water in a culvert near its outlet is effected similarly as for pressure culverts.

Computing Culvert Openings with Rectangular Cross Section

The discharge passed through the rectangular opening is computed from the following formula:

$$Q = \varphi\mu\, bh \sqrt{2g\,(H - \mu h)}$$

where:

φ — coefficient of velocity; for unstreamlined inlets $\varphi = 0.85$;
μ — coefficient of contraction; for rectangular openings $\mu = 0.60$;
b — width of culvert in m;
h — height of culvert in m;
g — acceleration due to gravity in m/sq sec.;
H — depth of elevated water counted from the culvert bottom in m.

The following formula for computing discharge is arrived at after substituting suitable values:

$$Q = 2.3\, bh \sqrt{H - 0.6\, h}$$

Computing Culvert Openings with Circular Cross Section

The discharge through a circular opening is computed from the formula:

$$Q = \varphi\mu \frac{\pi d^2}{4} \sqrt{2g(H - \mu d)}$$

where:

φ — coefficient of velocity equal to 0.85;
μ — coefficient of contraction; for a circular opening $\mu = 0.65$
d — diameter of culvert opening in m.

The following formula for computing the discharge is arrived at after substituting suitable values:

$$Q = 1.9\, d^2 \sqrt{H - 0.65\, d}$$

Having obtained the necessary values from these formulas, the clear span of the opening is determined by trials.

Complete Filling of Culvert Opening

The complete filling of a pressure opening appears when streamlined shapes of an inlet are adopted if the depth of a stream above it is $H > 1.4.h$. A culvert should be filled with water over its whole length, which may happen only when the slope of culvert I is smaller than the decrease in the friction I_t (losses of power per 1 m of culvert length):

$$I < I_t = \frac{Q^2}{F^2 C^2 R} = \frac{Q^2}{K_d^2}$$

because $F^2\, C^2\, R = K_d^2$ (K_d = modulus of discharge in an opening).

If the level of water in an outleading channel is higher by the magnitude Δh than the upper edge of the culvert, the level of elevated water above the culvert will also rise to the same height Δh, if:

$$\Delta h > \frac{v_d\, (v - v_d)}{g}$$

where:

v — water velocity in an opening in m/sec.;

v_d — water velocity below the culvert in m/sec.

Computing Culvert Openings with Rectangular Cross Section

The openings of culverts with rectangular cross sections and streamlined inlets are computed from the following formula:

$$Q = \varphi\, bh \sqrt{2g\,(H - h)}$$

where:

φ — coefficient of velocity; for streamlined openings $\varphi = 0.95$; the remaining denotations are the same as in former formulas.

The following is the water velocity near the inlet:

$$v = \frac{Q}{bh}$$

Computing Culvert Openings with Circular Cross Section

The volume of water flowing through an opening with a circular cross section may be computed from the formula:

$$Q = \varphi \frac{\pi d^2}{4} \sqrt{2g\,(H - d)}$$

The velocity at the culvert outlet amounts to:

$$v = \frac{4Q}{3.14\, d^2}$$

Computing Culvert Openings by Means of Diagrams and Tables

Tables and diagrams have been elaborated to facilitate the work of determining the openings.

An approximate computation of circular pressure culvert openings can be made for a free weir by means of the Bogomolov diagram. Among many tables elaborated for computing the clear span of openings, the most to be recommended are the Andreyev-Boldakov Tables.

Computing Culvert Openings by Means of the Bogomolov Diagram

The Bogomolov diagram can serve for the approximate determination of the clear span of circular pressureless openings (Fig. 131).

The heights of a culvert filling $k = \frac{H}{d}$ and the coefficients of velocity φ are shown in the diagram on the right hand side, the volumes of discharge Q — from the top downwards, and the curve of the magnitude of the filling angle δ — from the bottom upwards. The method of using the diagram is explained by an example.

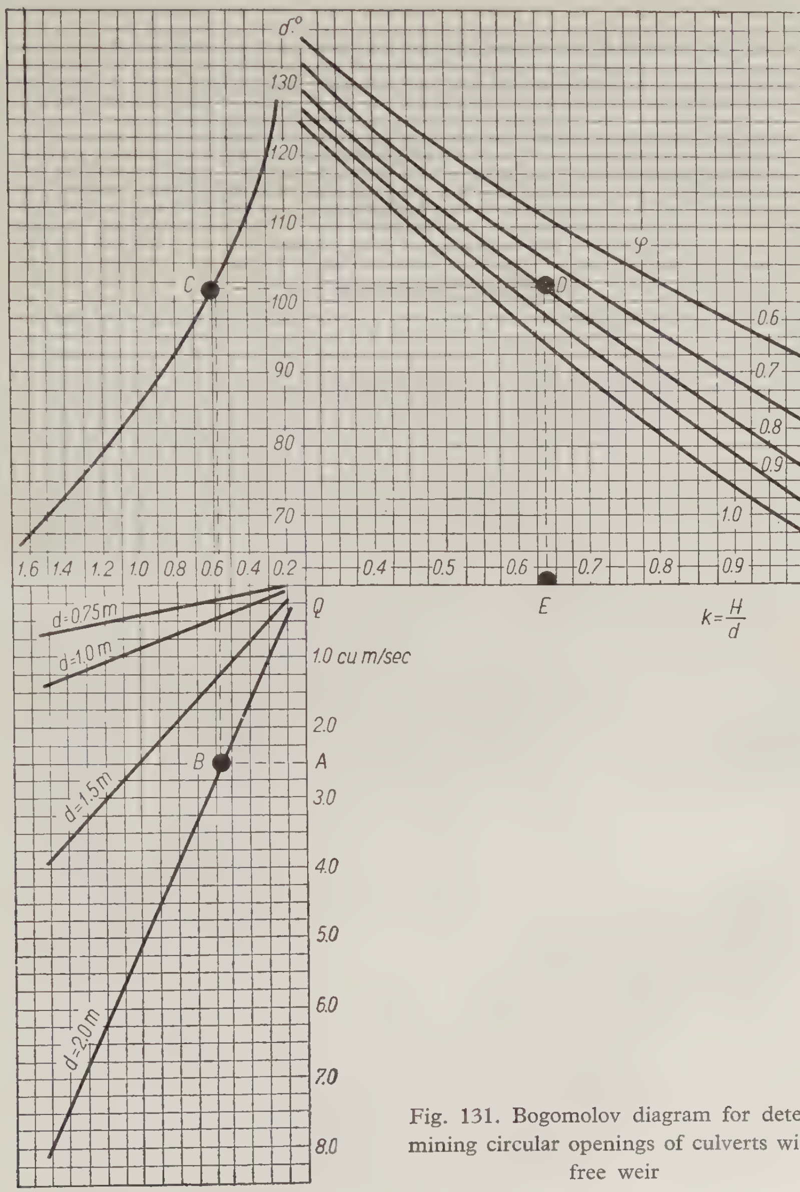


Fig. 131. Bogomolov diagram for determining circular openings of culverts with free weir

Example

A pipe diameter is to be selected and the depth of water at the outlet to be determined for the discharge $Q = 2.5$ cu m/sec., if the coefficient of velocity $\varphi = 0.8$.

Solution: From the point A corresponding with the discharge $Q = 2.5$ cu m/sec, a horizontal line is traced to the left, bisecting at the point B the line corresponding with the diameter of a pipe $d = 2.0$ m since we intend to apply such a diameter of opening. Subsequently, from the point B, a vertical line is traced upwards, which determines point C at the point of intersection with the curve of filling δ. From the point C, a horizontal line is then traced to the right, bisecting the curve $\varphi = 0.8$, at point D. The vertical line drawn downwards from the last-named point intersects the axis of abscissae k at point E.

The following value is found for the point E on the axis of abscissae

$$k = \frac{H}{d} = 0.630$$

hence:

$$H = k\,d = 0.630 \times 2.0 = 1.26 \text{ m}$$

The computations show that a 2.5 cu m/sec discharge can pass through an opening 2.0 m in diameter; the depth of water H above the structure will amount to 1.26 m (for $d = 1.25$ m, H will amount to 1.35 m).

Computing Culvert Openings by Means of Tables

The Andreyev-Boldakov tables, convenient in use and giving sufficiently accurate results, are recommended for computing culvert openings.

Appropriate depths of the elevated water H and velocities v, separately for unstreamlined *I* and streamlined *II* inlets, are given in these tables for individual discharges and sizes of openings.

In these tables, the coefficient of losses at the inlet $\xi = 0.40$ is adopted for ordinary inlets, coefficient of velocity $\varphi = 0.85$, coefficient of contraction of the surface at the opening entrance $\mu = 0.65$, and coefficient of contraction of the height at the entrance to the rectangular opening $\mu_\omega = 0.65$ and the circular opening $\mu_\omega = 0.60$.

In preparing the tables, it was assumed that openings are pressureless if $H \leqslant 1.2$ h, or for circular openings, $H \leqslant 1.2\,d$, while if $H > 1.2\,h$ or $H > 1.2\,d$, they are pressure openings but only with a part of the appropriate cross section.

The values of coefficients: $\xi = 0.10$, $\varphi = 0.95$ and $\mu = \mu_\omega = 1$ were adopted for streamlined openings *II*.

Openings with streamlined inlets are pressureless if $H \leqslant 1.4\,h$ ($H \leqslant 1.4\,d$),

while if $H > 1.4\, h$ ($H > 1.4\, d$) they are pressure ones with the full appropriate cross section.

Example 1

To pass the discharge $Q = 1.8$ cu m/sec a circular pressure opening is to be determined with admissible filling of the opening $k = 0.8$.

Determine by tables:

(a) the required diameter of the opening,

(b) the depth at the outlet of a culvert designed at slopes $I = 0$ and $I = 0.01$.

By means of Table 90, we establish that a circular opening with diameter $d = 1.5$ m should be applied to pass the $Q = 1.8$ cu m/sec discharge at $H = 1.08$ m and $v = 2.2$ m/sec, because the filling for such an opening k amounts to:

$$k = \frac{1.08}{1.50} = 0.72 < 0.8$$

The depth of water at the outlet of the culvert is computed by means of Table 88. The critical depth at the outlet will appear at the culvert bottom slope $I = 0$. The critical function in this case will be equal to:

$$\frac{Q^2}{g\, d^5} = \frac{1.8^2}{9.81 \times 1.5^5} = \frac{3.24}{74.5} = 0.043$$

From Table 88 it appears that such a value of the critical function corresponds with the depth of filling:

$$\frac{h_k}{d} = 0.45$$

The following depth at the outlet is computed by means of this value:

$$h_k = 0.45\, d = 0.45 \times 1.5 = 0.68 \text{ m}$$

A depth lower than the critical depth will appear at the culvert outlet for slope $I = 0.01$, which is probably greater than the critical slope.

The modulus of the existing discharge should be computed from the following formula to establish this depth:

$$K_o = \frac{Q}{\sqrt{I}} = \frac{1.8}{0.1} = 18 \text{ cu m/sec}$$

as also the modulus of discharge capacity of the entire opening:

$$K_d = 24\, d^{8/3} = 24 \times 1.5^{8/3} = 70.8 \text{ cu m/sec}$$

Table 89

Computing the Width of Rectangular Openings

Height of an opening in m	1.00		1.50		2.00		2.50		3.00		3.50		4.00	
Shape of inlet	I	II	I	II	I	II	I	II	I	II	I	II	I	II
Discharge Q in cu m/sec for 1 m of the width of opening	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.4	0.44	0.40	0.44	0.40	0.44	0.40	0.44	0.40	0.44	0.40	0.44	0.40	0.44	0.40
	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
0.6	0.58	0.52	0.58	0.52	0.58	0.52	0.58	0.52	0.58	0.52	0.58	0.52	0.58	0.52
	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8
0.8	0.70	0.64	0.70	0.64	0.70	0.64	0.70	0.64	0.70	0.64	0.70	0.64	0.70	0.64
	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
1.0	0.82	0.74	0.82	0.74	0.82	0.74	0.82	0.74	0.82	0.74	0.82	0.74	0.82	0.74
	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1
1.2	0.93	0.84	0.93	0.84	0.93	0.84	0.93	0.84	0.93	0.84	0.93	0.84	0.93	0.84
	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
1.4	1.02	0.92	1.02	0.92	1.02	0.92	1.02	0.92	1.02	0.92	1.02	0.92	1.02	0.92
	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
1.6	1.11	1.00	1.11	1.00	1.11	1.00	1.11	1.00	1.11	1.00	1.11	1.00	1.11	1.00
	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
1.8	1.21	1.09	1.21	1.09	1.21	1.09	1.21	1.09	1.21	1.09	1.21	1.09	1.21	1.09
	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
2.0	1.35	1.17	1.31	1.17	1.31	1.17	1.31	1.17	1.31	1.17	1.31	1.17	1.31	1.17
	3.2	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
2.5	1.76	1.36	1.51	1.36	1.51	1.36	1.51	1.36	1.51	1.36	1.51	1.36	1.51	1.36
	4.0	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9
3.0	2.27	1.51	1.68	1.52	1.68	1.52	1.68	1.52	1.68	1.52	1.68	1.52	1.68	1.52
	4.8	3.0	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
3.5	2.88	1.69	1.93	1.69	1.88	1.69	1.88	1.69	1.88	1.69	1.88	1.69	1.88	1.69
	5.6	3.5	3.8	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
4.0	3.56	1.90	2.23	1.85	2.05	1.85	2.05	1.85	2.05	1.85	2.05	1.85	2.05	1.85
	6.4	4.0	4.3	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
4.5	4.24	2.14	2.59	2.01	2.23	2.01	2.23	2.01	2.23	2.01	2.23	2.01	2.23	2.01
	7.2	4.5	4.8	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
5.0		2.41	2.97	2.13	2.39	2.15	2.39	2.15	2.39	2.15	2.39	2.15	2.39	2.15
		5.0	5.4	3.6	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7
5.5		2.71	3.40	2.25	2.54	2.30	2.54	2.30	2.54	2.30	2.54	2.30	2.54	2.30
		5.5	5.9	3.7	4.4	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
6.0		3.03	3.87	2.40	2.89	2.44	2.70	2.44	2.70	2.44	2.70	2.44	2.70	2.44
		6.0	6.4	4.0	4.8	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9
6.5		3.39	4.38	2.56	3.18	2.58	2.86	2.58	2.86	2.58	2.86	2.58	2.86	2.58
		6.5	7.0	4.3	5.2	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
7.0		3.77		2.73	3.50	2.73	3.02	2.73	3.02	2.73	3.02	2.73	3.02	2.73
		7.0		4.7	5.6	4.1	4.1	4.1	4.1	4.1	4.1	4.1	4.1	4.1
7.5				2.91	3.81	2.85	3.20	2.85	3.16	2.85	3.16	2.85	3.16	2.85
				5.0	6.0	4.2	4.8	4.2	4.2	4.2	4.2	4.2	4.2	4.2
8.0				3.11	4.18	2.90	3.43	2.98	3.30	2.98	3.30	2.98	3.30	2.98
				5.3	6.4	4.3	5.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2
9.0				3.55	4.96	3.14	3.95	3.22	3.56	3.22	3.56	3.22	3.56	3.22
				6.0	7.2	4.5	5.8	4.4	4.4	4.4	4.4	4.4	4.4	4.4
10				4.01		3.41	4.48	3.46	3.90	3.46	3.83	3.46	3.83	3.46
				6.7		5.0	6.4	4.6	5.4	4.6	4.6	4.6	4.6	4.6
11				4.55		3.71	5.11	3.68	4.35	3.68	4.07	3.68	4.07	3.68
				7.3		5.5	7.1	4.7	5.9	4.7	4.7	4.7	4.7	4.7
12						4.03		3.80	4.80	3.88	4.34	3.88	4.30	3.88
						6.0		4.8	6.4	4.9	5.5	4.9	4.9	4.9
13						4.39		4.02	5.31	4.10	4.72	4.10	4.55	4.10
						6.5		5.2	7.0	5.0	6.0	5.0	5.0	5.0

Table 89 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
14						4.75		4.28		4.24	5.12	4.30	4.78	4.30
						7.0		5.6		5.1	6.5	5.1	5.1	5.1
15								4.53		4.41	5.56	4.52	5.05	4.52
								6.0		5.2	6.9	5.3	6.0	5.3
16								4.82		4.61	6.04	4.70	5.42	4.70
								6.4		5.3	7.4	5.4	6.4	5.4
17								5.11		4.81		4.90	5.79	4.90
								6.8		5.7		5.5	6.8	5.5
18								5.44		5.03		5.00	6.18	5.09
								7.2		6.0		5.5	7.2	5.6
19										5.26		5.17		5.29
										6.3		5.6		5.7
20										5.51		5.33		5.46
										6.7		5.7		5.8
21										5.77		5.53		5.62
										7.0		6.0		5.9
22												5.73		5.71
												6.3		5.9
23												5.94		5.87
												6.6		5.9
24												6.15		6.03
												6.8		6.0
25														6.20
														6.2
26														6.39
														6.5
27														6.58
														6.7
28														6.77
														7.0

The ratio of these moduli amounts to:

$$\frac{K_o}{K_d} = \frac{18}{70.8} = 0.254$$

We find from Table 88 that $\frac{h_o}{d} = 0.35$ for this value and, therefore, h_o is equal to:

$$h_o = 0.35\,d = 0.35 \times 1.5 = 0.53 \text{ m}$$

Comparing $h_o = 0.53$ m with $h_k = 0.68$ m we find that the slope $I = 0.01$ is really greater than the critical slope.

From Table 88, we take the following value of the ratio of moduli for $\frac{h_o}{d} = 0.35$:

$$\frac{W_o}{W_d} = 0.821$$

Since $W_d = 30.5\,d^{2/3} = 30.5 \times 1.5^{2/3} = 40$ m/sec then W_o amounts to:

$$W_o = 0.821 \times 40 = 32.8 \text{ m/sec}$$

The velocity at the outlet is equal to:

$$v = W_o \sqrt{I} = 32.8 \times 0.1 = 3.3 \text{ m/sec}$$

or it is greater than the velocity at the opening amounting to 2.2 m/sec.

Example 2

A circular cross section, partly a pressure cross section, is to be selected for passing the discharge $Q = 1.8$ cu m/sec provided that the velocity in the opening does not exceed 4 m/sec.

We establish by means of Table 90 that the following circular openings can be applied to pass the discharge $Q = 1.8$ cu m/sec:

(a) 0.90 m in diameter at $H = 2.03$ m and $v = 4.6$ m/sec.

(b) 1.00 m in diameter at $H = 1.59$ m and $v = 3.7$ m/sec.

Since the velocity in an opening cannot exceed 4 m/sec, we adopt the 1 m in diameter culvert.

The velocity at the outlet of the culvert is computed similarly as in example 1, i. e.:

$$K_o = \frac{Q}{\sqrt{I}} = 18 \text{ cu m/sec} \qquad K_d = 24\,d^{8/3} = 24 \text{ cu m/sec}$$

$$\frac{K_o}{K_d} = \frac{18}{24} = 0.75$$

The value $\frac{h_o}{d} = 0.64$, and depth at the outlet $h_o = d \times 0.64 = 0.64$ m, correspond with the latter ratio.

The ratio of moduli amounts to:

$$\frac{W_o}{W_d} = 1.11$$

Since $W_d = 30.5\ d^{2/3} = 30.5$ m/sec, therefore:

$$W_o = 30.5 \times 1.11 = 33.9 \text{ m/sec}$$

$$v = W_o \sqrt{I} = 33.9 \times 0.1 = 3.4 \text{ m/sec}$$

It results from the above that the velocity at the outlet $v = 3.4$ m/sec is somewhat lower than that at the inlet which is equal to 3.7 m/sec.

Example 3

A circular pressure opening working with its full cross section is to be selected to pass a discharge $Q = 1.8$ cu m/sec, provided that the velocity in the opening does not exceed 4 m/sec.

We establish by means of Table 90 that the following circular openings may be applied to pass the discharge $Q = 1.8$ cu m/sec through the culvert with a streamlined inlet:

(a) 0.75 m in diameter at $H = 1.63$ m and $v = 3.9$ m/sec,

(b) 0.90 m in diameter at $H = 1.38$ m and $v = 2.8$ m/sec.

As more economical, we adopt the opening with diameter of 0.75 m.

It is clear from this example that the adoption of a streamlined inlet makes it easier to diminish the diameter of the opening from $d = 1$ m (example 2) to $d = 0.75$ m.

The decrease in the friction for a circular culvert $d = 0.75$ m is determined from the formula:

$$I_t = \frac{Q^2}{K_d^2} = \frac{Q^2}{(24\, d^{8/3})^2} = \frac{1.8^2}{11.15^2} = 0.026$$

As appears from the above, the slope of the culvert I cannot be greater than 0.026.

For slope $I = 0.01 < I_t$ in a culvert $l = 20$ m, the additional elevation Δl of the level of the already elevated water will amount to:

$$\Delta l = l\,(I_t - I) = 20 \quad (0.026 - 0.01) = 0.32 \text{ m}$$

The depth of the elevation will then be equal to:

$$H_1 = H + \Delta l = 1.63 + 0.32 = 1.95 \text{ m}$$

The velocity at the outlet will amount to:

$$v = \frac{4Q}{\pi\, d^2} = \frac{4 \times 1.8}{3.14 \times 0.75^2} = \frac{7.2}{1.77} = 4.07 \text{ m/sec}$$

Table 90

Computing the Diameter of Circular Openings

Diameter of an opening in m	0.75		0.90		1.00		1.25		1.50		1.75		2.00	
Shape of inlet	I	II	I	II	I	II	I	II	I	II	I	II	I	II
Discharge Q in cu m/sec	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$	$\frac{H}{v}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.4	0.61 1.7	0.56 1.7	0.56 1.6	0.52 1.6	0.55 1.6	0.51 1.6	0.51 1.5	0.47 1.5	0.48 1.5	0.42 1.5	0.47 1.5	0.41 1.5	0.45 1.5	0.38 1.5
0.6	0.79 1.9	0.72 1.9	0.72 1.9	0.66 1.9	0.70 1.8	0.64 1.8	0.62 1.7	0.57 1.7	0.60 1.7	0.56 1.7	0.60 1.6	0.52 1.6	0.55 1.6	0.49 1.6
0.8	1.00 2.9	0.85 2.1	0.86 2.0	0.77 2.0	0.82 2.0	0.76 2.0	0.73 1.8	0.67 1.8	0.69 1.8	0.64 1.8	0.67 1.8	0.62 1.8	0.67 1.7	0.61 1.7
0.9	1.17 3.1	0.92 2.2	0.93 2.1	0.84 2.1	0.88 2.0	0.81 2.0	0.76 1.9	0.71 1.9	0.74 1.8	0.68 1.8	0.72 1.8	0.66 1.8	0.69 1.8	0.65 1.8
1.0	1.33 3.5	0.99 2.3	1.00 2.2	0.90 2.2	0.94 2.1	0.86 2.1	0.82 1.9	0.75 1.9	0.78 1.9	0.72 1.9	0.76 1.9	0.70 1.9	0.73 1.9	0.68 1.9
1.1	1.51 3.8	1.07 2.4	1.05 2.3	0.95 2.3	1.00 2.2	0.91 2.2	0.86 2.0	0.79 2.0	0.82 1.9	0.76 1.9	0.79 1.9	0.72 1.9	0.77 1.9	0.72 1.9
1.2	1.72 4.2	1.14 2.6	1.18 3.1	1.00 2.4	1.06 2.3	0.96 2.3	0.91 2.1	0.84 2.1	0.87 2.0	0.80 2.0	0.82 1.9	0.76 1.9	0.81 1.9	0.75 1.9
1.4	2.19 4.9	1.28 3.0	1.43 3.5	1.09 2.6	1.17 2.5	1.06 2.5	1.00 2.2	0.92 2.2	0.93 2.1	0.87 2.1	0.89 2.0	0.82 2.0	0.89 2.0	0.82 2.0

Table 90 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1.6	2.69 5.6	1.44 3.5	1.70 4.0	1.21 2.7	1.37 3.4	1.14 2.6	1.09 2.3	1.00 2.3	1.00 2.2	0.93 2.2	0.96 2.1	0.89 2.1	0.93 2.1	0.87 2.1
1.8		1.63 3.9	2.03 4.6	1.38 2.8	1.59 3.7	1.23 2.6	1.16 2.4	1.07 2.4	1.08 2.2	1.00 2.2	1.03 2.1	0.95 2.1	0.99 2.1	0.92 2.1
2.0		1.84 4.3	2.36 5.0	1.47 3.1	1.80 4.1	1.32 2.7	1.26 2.5	1.13 2.5	1.15 2.3	1.07 2.3	1.09 2.2	1.00 2.2	1.05 2.2	0.97 2.2
2.2		2.07 4.7	2.67 5.5	1.57 3.4	2.04 4.6	1.47 2.8	1.33 2.6	1.21 2.6	1.21 2.4	1.12 2.4	1.14 2.3	1.07 2.3	1.11 2.2	1.02 2.2
2.5		2.42 5.4	3.34 6.2	1.78 3.9	2.47 5.1	1.58 3.2	1.43 2.8	1.31 2.8	1.30 2.5	1.20 2.5	1.23 2.4	1.12 2.4	1.19 2.3	1.10 2.3
3.0				2.17 4.6	3.26 6.2	1.82 3.8	1.86 3.8	1.45 2.9	1.47 2.7	1.33 2.7	1.37 2.6	1.26 2.6	1.30 2.4	1.21 2.4
3.5				2.58 5.4	4.20 7.2	2.14 4.5	2.24 4.6	1.60 3.1	1.63 2.9	1.48 2.9	1.51 2.7	1.38 2.7	1.41 2.6	1.31 2.6
4.0				3.09 6.2		2.47 5.1	2.66 5.2	1.84 3.2	1.75 3.1	1.60 3.1	1.66 2.8	1.49 2.8	1.53 2.7	1.43 2.7
4.5						2.87 5.7	3.26 5.9	1.98 3.7	2.07 4.2	1.71 3.2	1.75 2.9	1.59 2.9	1.65 2.8	1.53 2.8
5.0						3.27 6.3	3.70 6.5	2.17 4.0	2.38 4.6	1.83 3.3	1.87 3.0	1.69 3.1	1.75 2.9	1.61 2.9
5.5						3.76 6.9	4.25 7.2	2.37 4.5	2.67 5.0	1.95 3.4	1.98 3.2	1.80 3.2	1.86 3.0	1.71 3.0

Table 90 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
6.0							5.00 7.8	2.58 4.8	2.99 5.5	2.09 3.6	2.08 3.4	1.89 3.4	1.97 3.1	1.79 3.1
6.5								2.82 5.2	3.32 5.9	2.27 3.7	2.38 4.5	1.97 3.4	2.06 3.2	1.88 3.2
7.0								3.09 5.7	3.77 6.3	2.40 4.0	2.67 4.8	2.06 3.5	2.16 3.3	1.97 3.3
7.5								3.34 6.1	4.16 6.8	2.52 4.2	2.87 5.1	2.17 3.6	2.26 3.4	2.07 3.4
8.0								3.64 6.4	4.54 7.2	2.64 4.5	3.12 5.4	2.27 3.7	2.34 3.6	2.13 3.6
8.5								3.91 6.8	5.09 7.7	2.79 4.8	3.36 5.7	2.38 3.7	2.38 3.6	2.18 3.6
9.0								4.22 7.2	5.60 8.2	2.93 5.1	3.60 6.1	2.53 3.8	2.66 4.6	2.29 3.7
9.5								4.60 7.7	6.07 8.6	3.10 5.4	3.94 6.4	2.64 4.0	2.86 4.9	2.36 3.7
10										3.29 5.6	4.26 6.7	2.73 4.2	3.07 5.1	2.44 3.8
11										3.68 6.2	4.95 7.5	2.93 4.6	3.46 5.6	2.62 3.9
12										4.12 6.8	5.67 8.1	3.14 5.1	3.83 6.1	2.74 4.0

Table 90 (continued)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
13										4.52 7.3	6.42 8.8	3.42 5.5	4.25 6.6	2.96 4.1
14										4.99 7.9	7.37 9.5	3.70 5.9	4.76 7.1	3.10 4.4
15										5.51 8.4		4.02 6.3	5.29 7.6	3.28 4.7
16												4.30 6.7	5.82 8.1	3.40 5.0
17												4.61 7.0	6.47 8.7	3.60 5.4
18												4.92 7.6	7.08 9.2	3.80 5.7
19												5.32 8.0	7.81 9.8	4.04 6.0
20												5.69 8.4		4.27 6.4
21												6.12 8.8		4.50 6.7
22												6.64 9.3		4.74 7.0
23														4.98 7.2

CHAPTER X

TAKING WATER ACCUMULATION INTO ACCOUNT IN COMPUTING OPENINGS OF SMALL STRUCTURES

1. Accumulation of Water Above the Structure

Water running off the surface of a catchment basin usually accumulates above the inlets of openings the clear span of which is smaller than the width of a stream and forms an artificial lake. The quantity of water in such lake primarily depends on the configuration of the terrain and the height of water elevation (Fig. 132).

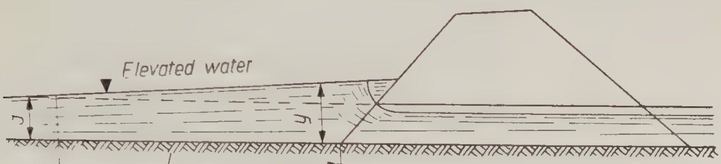


Fig. 132. Scheme of Water Accumulation above a Structure

The accumulation of water in small streams may considerably diminish the discharge passed per second through a structure, thus facilitating the reduction of the clear span of the opening of such structure.

By reason of accumulation, the maximum discharge passed per second through an opening may even be reduced to one tenth of the maximum discharge in the appropriate cross section. For this reason, accumulation should always be taken into account in small catchment basins, exceptions here are cases in which forming a reservoir is not advisable in view of losses and damages which may be caused by a partial or complete flooding of the neighboring areas.

In taking accumulation into account, the volume of a reservoir and the flow should above all be determined. The volume of a reservoir is determined by means of contour lines on a map or by an approximate method, assuming that the water surface in a reservoir has the shape of a parabola $\frac{y}{i}$ meters long. With such an assumption, the area of the reservoir surface A and the volume W will amount respectively to:

$$A = \frac{2}{3} b \frac{y}{i}$$

$$W = \frac{1}{3} y A = \frac{y}{3} \cdot \frac{2by}{3i} 1000 \approx 220 \frac{by^2}{i_1}$$

where:

b — width of the surface of the reservoir above a structure;
y — depth of reservoir;
i_1 — slope of stream channel, in per mil;
i — unit slope of channel per 1 m of length.

The graph of the course of the discharge, called a hydrograph (Fig. 133), serves for computing the total volume of discharge.

2. Methods of Taking into Account the Accumulation of Water

Methods of taking accumulation into account have been presented by Boldakov, Sokolovskii, Dadenkov, Chegodaev and others.

For computing openings and taking into account water accumulation, it is necessary to draw a hydrograph of discharge.

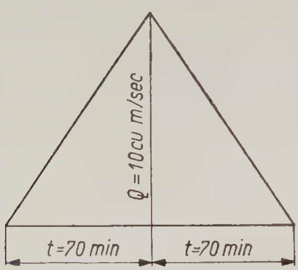


Fig. 133. Hydrograph of a discharge

A hydrograph may have various shapes depending on the character of a catchment basin and the type of the runoff. Establishing the real shape of a hydrograph is usually impossible. For small catchment basins, Boldakov and Ogevskii recommend hydrographs shaped like an isosceles triangle, in which the duration of rise is equal to the duration of subsidence.

Such a shape of hydrograph drawn for small catchment basins is approximately equal to the real one. The difference consists only in the fact that the real hydrograph has the shape of a triangle with a flattened vertex and slightly concave sides. The duration of the rise of a discharge is 10 to 30 percent less than the duration of subsidence.

If the runoff from a catchment basin is computed by means of isochrones, the shapes of the hydrographs may differ considerably, depending on the area between the isochrones and the duration of rains. In such a case, the all-over hydrograph consists of separate hydrographs for individual parts of a catchment basin.

The methods of taking into account accumulation — presented by the authors mentioned above — must therefore be based on an approximate shape of a hydrograph. For this reason, exaggerated accuracy in the remaining computations is not appropriate and certain simplifications facilitating the work may be permitted.

On this assumption, the method of taking accumulation into account has

been presented by the author of this book without establishing flows at particular intervals of time, as recommended by, for instance, Boldakov[6]. In this case, only the total volume inflowing to the structure and the total capacity of a reservoir at the maximum height to which it may be filled is computed. The admissible greatest depth of a reservoir should be adopted depending on local conditions.

The order of computing an opening by the Jarocki method is clear from the numerical example given below.

Example 1

Compute the size of opening, water accumulation being taken into account, for an adopted maximum discharge of 10 cu m/sec. The area of the catchment basin up to the bridge passageway F 2.5 sq km, and the slope of the stream channel is equal to 0.005 (Fig. 132). Duration of a water rise t is equal to the duration of the subsidence $t = 70$ min (Fig. 133).

We adopt a circular pressure opening with an unstreamlined inlet. The following formula serves for computing the discharge in such an opening:

$$Q = 1.9\, d^2 \sqrt{H - 0.65\, d}$$

The following is the order of further computations:

(1) the hydrograph of the discharge is drawn in the form of a triangle the height of which is equal to 10 cu m/sec and base $2t = 140$ minutes;

(2) the total volume of afflux is computed as equal to the surface area of the hydrograph:

$$W_d = \frac{2tQ}{2}\, 60 = \frac{140 \times 10 \times 60}{2} = 42000 \text{ cu m}$$

(3) the admissible depth of the reservoir filling $y = 2.0$ m is adoptad for given conditions;

(4) the capacity of the reservoir is determined for this depth from the following formula:

$$W_z = \frac{220\, by^2}{i_1} = \frac{220 \times 180 \times 2^2}{5} = 31680 \text{ cu m}$$

where b — width of reservoir established from the map or in the field (in the example $b = 180$ m);

[6]) The Boldakov method is described in "Hydrologiczne i hydrauliczne obliczenia przepustów i małych mostów" (Hydrological and Hydraulic Computations of Small Bridges and Culverts) by W. Jarocki, published by The National Hydro-Meteorological Institute, Warsaw in 1953 which has been translated and published by the CIINTE at the request of the National Science Foundation in Washington in 1962.

(5) the water volume W_0 which is to pass through the opening is computed from the equation:

$$W_o = W_d - W_z = 42000 - 31680 = 10320 \text{ cu m}$$

(6) the discharge which should pass through the opening after the reservoir is filled is established from the formula:

$$Q_1 = \frac{W_o}{60\,t} = \frac{10320}{60 \times 70} = 2.46 \text{ cu m sec}$$

this equation involves the assumption of a uniform flow of water in time t, which is not absolutely correct, but can be admitted as facilitating the computations;

(7) the clear span of an opening necessary to pass the computed discharge Q_1 is determined by the formula presented above:

$$Q_1 = 1.9\, d^2 \sqrt{y - 0.65\, d}$$

The diameter of the opening d, which for this case is equal to $d = 1.05$ m is computed from this equation by a trial:

$$2.46 = 1.90 \times 1.05^2 \sqrt{2.0 - 0.65 \times 1.05}$$

To pass the same discharge 10 cu m/sec — without taking into account accumulation $\frac{10}{2.46} \approx 4$ openings of this diameter should be applied.

Example 2

Compute the size of the opening of a culvert with a rectangular cross section, with and without taking into account the accumulation of water of summer rains in a catchment basin of area $F = 64$ sq km, if $Q_p = 56$ cu m/sec, $h = 17$ mm, and $t = 30$ min.

The computation of the values Q_p, h and t is as given in Chapter V (example for computing maximum discharge by the DORNII formula).

On the basis of Table 89, we assume, that the rectangular opening 1 m wide and 3 m high passes the discharge $Q_1 = 14$ cu m/sec at a depth of the elevated water $H = 4.24$ m, and velocity $v = 5.1$ m/sec.

A culvert with the following width of opening should be used for the existing discharge $Q = 56$ cu n/sec disregarding the accumulation:

$$b = \frac{Q_p}{Q_1} = \frac{56}{14} = 4 \text{ m}$$

When the accumulation is taken into account, the discharge passed through the opening may be computed from the formula:

$$Q = Q_p \left[1 - \left(\frac{W_z}{W_p}\right)^m\right]$$

where:

Q — volume of water flowing through the opening in cu m/sec;

Q_p — discharge with a defined probability of occurrence inflowing from the catchment basin in cu m/sec;

W_z — capacity of the reservoir with the adopted depth in cu m;

W_p — flow from the catchment basin, in cu m;

m — coefficient of accumulation which is equal to: for catchment basins without lakes and marshes, $m = 0.75$, for catchment basins crossed by a road above the cross section under computation and containing a small number of lakes and marshes, $m = 0.5$,
for catchment basins with a large number of lakes and marshes, $m = 0.25$.

The volume of the reservoir W_z is computed from the formula:

$$W_z = \frac{220\, b\, h^2}{i}$$

where:

b — width of a reservoir, in m,

h — depth of a reservoir, in m,

i — slope of the bed of a stream channel, in per mil.

As in the example presented, at depth of elevated water $h = 4.24$ m, and slope of bed of stream channel $i = 2.5°/_{00}$, width of reservoir $b = 400$ m; therefore, the capacity of the reservoir amounts to:

$$W_z = \frac{220 \times 400 \times 4.24^2}{2.5} = 632\,900 \text{ cu m}$$

The flow from the catchment basin is equal to:

$$W_p = 1000\, Fh = 1000 \times 64 \times 17 = 1\,088\,000 \text{ cu m}$$

The volume of water flowing through the opening, taking accumulation into account, amounts to:

$$Q = Q_p \left[1 - \left(\frac{W_z}{W_p}\right)^m\right] = 56 \left[1 - \left(\frac{632.9}{1088}\right)^{0.75}\right] = 56 \times 0.522 = 29.2 \text{ cu m/sec}$$

The width of the opening, taking accumulation into account, is equal to:

$$b = \frac{29.2}{14} \approx 2 \text{ m}$$

It is clear from the examples presented above that taking accumulation into account affords considerable economy and, therefore, it cannot be omitted in computing openings of small structures.

CHAPTER XI

CLOSING REMARKS

This work makes it clear that various kinds of investigations and studies should be conducted and several diagrams and auxiliary calculations prepared in order to compute the opening of a bridge or culvert. This will facilitate the determination of a more accurate value of the maximum discharge and dimensions of an opening necessary to pass the existing discharge.

The openings of bridges and culverts should be computed for discharges with defined probability of occurrence.

The computation of discharges of this type on large and average rivers, usually presents no difficulties. On larger rivers, there is usually a sufficient number of hydrometric observations which facilitate the computation of a valid discharge by known methods.

Much more difficult, however, is the determination of maximum discharges with defined probability of occurrence on small rivers, where observations and measurements have not been conducted at all or were only few in number.

On such streams, the maximum discharges are usually computed by analogy or by empirical formulas.

In an effort to obtain the most accurate possible results, the authors of these formulas, introducing various coefficients, have taken into account as many as possible of the factors exerting influence on the discharge.

It is desirable that the empirical formulas should be the most accurate possible. Introducing an excessive number of coefficients diminishes the value of such formulas since it complicates the computations and, at the same time makes it possible for designers to commit errors when selecting appropriate magnitudes of such coefficients. Besides, the influence of certain coefficients is questionable and difficult to characterize and, therefore, it can be inappropriately applied in the derivation of a formula.

Therefore, the actual factors exerting a basic influence on discharge should be taken into consideration.

The analysis of certain existing empirical formulas for computing maximum discharges are to be found in the works dedicated to hydrology. The analysis proves that some of these formulas give erroneous results, although they sometimes include several, and even some dozens of coefficients.

These errors are mostly due to the inadequately selected form of a formula, the coefficient of which was derived on the basis of an insufficient number of observations.

It is clear that not always are the best results given by formulas taking into account a large number of coefficients, which sometimes only complicate the computations.

The formulas derived should be simple and convenient in use. The computation of the maximum discharges is easy and rapid if performed by means of such formulas. This is of great importance in practice in view of the development of transport and the consequent necessity to compute large numbers of bridge and culvert openings for roads and railroads.

If these formulas are derived for small areas, the results of computations made without introducing a large number of coefficients will probably differ only slightly from the real values. Note that empirical formulas are considered by hydrologists as sufficiently accurate if they yield values differing from the real values within limits of about $\pm$ 30 percent.

As already indicated, determining maximum discharges with an adopted probability of occurrence is much easier in relation to large and average rivers than small streams.

On the other hand, the situation is otherwise as regards computing openings as concerning hydraulics. On large rivers, there appear large quantities of water, which may not always find easy passage through the bridge opening. The size of openings required on such rivers depends to a considerable extent on local conditions, which may in various ways affect the results of computations.

For this reason, it is difficult to give any guiding rules for use in individual cases. Therefore, the following operations should be carried out for the hydraulic computation of bridge openings on large and average rivers:

(a) determine the water current velocity admissible in the main channel under the bridge, depending on the type of soil;

(b) compute the discharge section area of a river and water velocities in a free channel, for the water stage adopted in computations;

(c) select the necessary number of bridge piers and appropriate coefficient of contraction, and also determine the area occupied by such piers;

(d) compute the necessary discharge section area and select it in the river cross section under the bridge, if necessary taking into account channel scour and artificial deepening of the bottom;

(e) determine the water movement velocity;

(f) establish the magnitude of an elevation above the bridge;

(g) compute the size of the opening.

Several variants of computations should be conducted with different coefficients of scour, with and without bottom deepening, etc. To find the most advantageous variant a graphical method should be used.

It is advisable in the interests of clarity to present the computation in the form of an assembled list, embracing the cross section of a river, the character-

istic water stages, depths of the scour, artificial deepening, type of soil, etc. being indicated.

The hydraulic computation of openings on small streams is easier than on large rivers.

For a rational design, hydraulically and economically speaking, of an opening of a small bridge or culvert, it is necessary:

(a) to adopt rectangular cross sections of openings, for large discharges and circular cross sections for lower discharges;

(b) to make every effort that a pressure opening will be able to work;

(c) to adopt a streamlined shape of inlets of submerged openings, so that they may work with the full cross section;

(d) to slant the wings of inlets and outlets of openings at an angle of about 30° to the axis of the culvert;

(e) to adopt the water velocity admissible in an opening not greater than 7 m/sec on account of the channel revetment below the outlet; in computing the structure of openings for an extreme stage, up to 15 m/sec velocity in an opening can be adopted;

(f) in a catchment basin area up to 100 sq km, water accumulation above the structure should be taken into account;

(g) the magnitude of small openings should be computed by the Andreev -Boldakov tables or formulas differently for free and submerged openings and for partial and complete filling.

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